DESIGN AND CONSTRUCTION OF LARGE-PANEL CONCRETE STRUCTURES

Contract No. H-2131R

report 3

Wall Panels: Analysis and Design Criteria

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ABSTRACT

This report investigates and delineates suitable methods of analysis and design of large structural wall panels for adequate strength and stability against normal loadings defined in Report 1, consisting of both vertical and horizontal forces. Design methods are outlined which take into account special considerations such as load concentrations, eccentricities of vertical loads, effects of slenderness, effects of openings, and effects of edge restraints. The design methods are compared with available experimental data for bearing walls and found to be satisfactory.

Finally, as an appendix to the report, illustrative numerical examples are given in detail.

This report is the third in a series of reports leading to the development of a "Recommended Practice for the Design and Construction of Large Panel (LP) Structures." This report deals with the analytical considerations and design methods for structural wall panels to resist normal loadings defined in Report 1. The following factors related to the structural behavior of wall panels are also examined:

- types,
- production techniques,
- geometry,
- tolerances
- handling stresses,
- loadings,
- eccentricities, and
- reinforcing.

Acceptable methods are outlined to take into account:

- concentrated vertical forces,
- openings,
- slenderness,
- thermal effects,
- vertical edge restraints, and
- end splitting.

Wall panels requiring uniformly distributed vertical reinforcement for either strength or serviceability should be designed as "Uniformly Reinforced Wall Panels". Based on the tradition of cast-in-place walls, the minimum amount of such uniformly distributed reinforcement is given in the ACI 318-71 Code as 0.15% for vertical reinforcement and 0.25% for horizontal reinforcement of the cross-sectional area. The PCI report on Precast Concrete Bearing Wall Buildings gives the minimum amount of such uniformly distributed reinforcement for precast walls as 0.10% of the gross cross-sectional area in both directions. The PCI report considers a lower minimum reinforcement sufficient for precast wall panels since such precast panels have little restraint at the edges during the curing and storage stages and, therefore, will not build up tensile shrinkage stresses as high as those in a cast-in-place walls.

Introduced in this report is the concept of "Peripherally Reinforced" wall panels. Such walls should have horizontal reinforcement at the top and bottom of the panel as well as minimum vertical tensile ties. The axial load-carrying capacity of such wall panels can be computed according to the procedures outlined in Chapter 14 of the ACI 318-71 Code. It should be noted that these procedures are also in conformance with the provisions of ACI 322 (Building Code Requirements for Structural Plain Concrete). The use of peripherally reinforced wall panels is recommended only in situations where the following conditions are satisfied:

(a) panels are produced in precast plants with good quality control,

(b) the panel is not subjected to out-of-plane bending during removal from the form,

(c) horizontal forces perpendicular to the plane of the wall panel are insignificant,

(d) the resultant of all vertical forces is within the kern of the section.

The appendix to this report includes a brief summary of European methods of wall design as well as illustrative numerical examples for the design of uniformly reinforced and peripherally reinforced wall panels.

OVERALL PROGRAM OBJECTIVES

The term "large panel" (LP) concrete structure is used to describe a structural system composed of precast wall panels with floors and roofs of precast panels or planks (Fig. 1). These prefabricated component buildings can be considered to be the industrialized form of conventional cast-in-place structural wall (egg crate) construction. Large panel buildings are differentiated by the general arrangement (Fig. 2) of load-bearing walls:

(a) Cross wall system: in this most prevalent system, the load-bearing cross walls are perpendicular to the longitudinal axis of the building.

(b) Spine wall system: the load-bearing walls are parallel to the longitudinal axis of the building.

(c) Mixed systems: a combination of cross wall and spine wall systems.

In most LP systems, the walls transfer their loads directly to the substructure without an intermediate frame. This form of construction restricts open plans at any level and is thus most typically suited for multistory housing where walls with a degree of mass usually have to be provided between apartments for fire safety provisions and reduction in noise transmission. Construction types considered under this investigative program include solid, sandwich, ribbed, hollow core and composite wall panels and solid, hollow core, or ribbed floor units with or without cast-in-place topping. Elements are either prestressed or conventionally reinforced.

The overall program objective is to develop minimum criteria for the design and construction of large panel structures. These criteria are being developed to ensure the structural safety and serviceability of LP residential buildings, while also providing minimum performance requirements to be used by designers and developers of other innovative systems. The development of the criteria will also expand the knowledge of design and construction of prefabricated large panel structures to a level comparable with that existing for conventional cast-in-place concrete or steel structural systems.

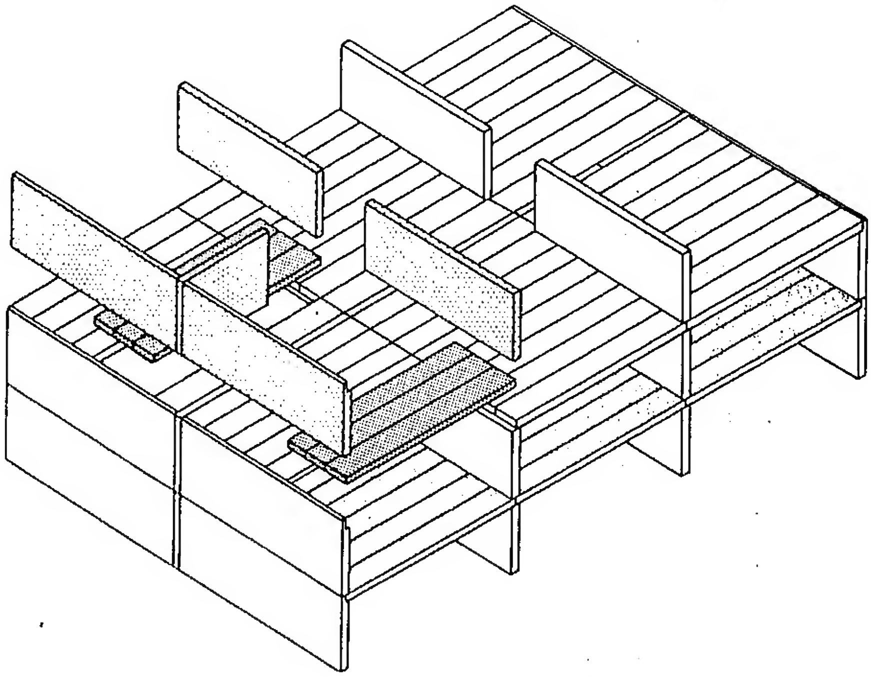
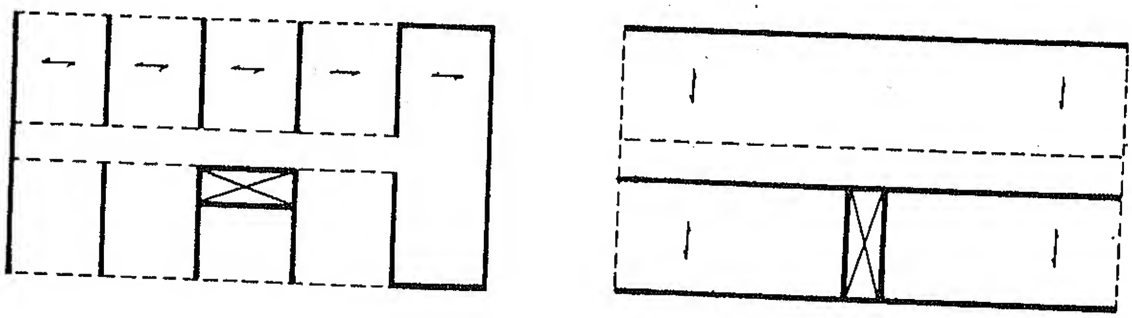


Fig. 1 Isometric View of Idealized Large Panel Structure



(a) Cross Wall System

(b) Spine Wall System

Fig. 2 Idealized Plan Arrangements of Structural Wall Panels in Large Panel Structures

3.1 INTRODUCTION

A typical large panel (LP) structure (Fig. 1) is made up of a substructure, wall panels (structural and nonstructural), and floor and roof panels. Wall panels are the principal vertical load-carrying elements since they transfer loads from the floor and roof panels to the substructure without an intermediate frame.

The cost of an LP structure is significantly affected by the number, size, and shape of its wall panels. Current production techniques for wall panels primarily include battery mold, tilt table, and flat bed casting. These differ from "assembly line" type casting beds used for hollow core floor panels. This dissimilarity in the casting process invariably makes the wall panel a relatively expensive structural element. As a result, to create an economical LP structural system the engineer should pay special attention not only to optimizing the layout of the wall panels, but also to the design of the elements themselves.

To effectively use the structural wall panel it is necessary to develop a rational method for its analysis and design. Such a procedure should be based on a study of the general characteristics of wall panels and should include an investigation of all critical analytical and design considerations.

3.2 GENERAL CHARACTERISTICS

Figure 2 shows idealized plan arrangements of structural wall panels in LP buildings. To better understand the behavior of wall elements under a system of applied vertical and horizontal forces, the following general factors are discussed and evaluated in detail:

- types,
- production techniques,
- geometry,
- tolerances,
- handling stresses,
- loadings,
- eccentricities, and
- reinforcing.

3.2.1 Types

Structural wall panels can be categorized on the basis of three characteristics: (1) cross-section, (2) function, and (3) location.

Cross-section characteristics describe the cross section of the wall panel. Figure 3 illustrates the following, qualitatively described, typical cross sections of wall panels:

- solid
- solid ribbed
- composite sandwich
- noncomposite sandwich
- hollow core

Function characteristics describe the principal structural purpose of the wall panel. For example, a load-bearing wall describes an element transmitting vertical loads from adjoining elements to the substructure. A nonload-bearing wall defines an element that provides space separation and, on occasion, additional lateral rigidity. Such walls usually do not transmit vertical loads.

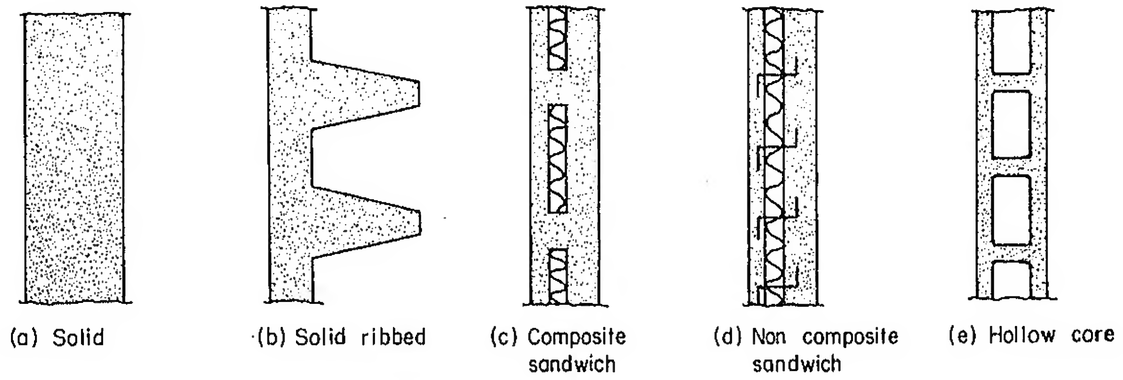


Fig. 3 Typical Sections of Structural Wall Panels Used in Current Practice

Location characteristics describe the position and direction of the wall panel with reference to the plan arrangement. For example, an interior wall refers to a panel located within the building floor plan, while a flank wall refers to a panel located along the perimeter of the structure. Figure 4 shows the use of the cross-section, function, and location characteristics in categorization of wall panels.

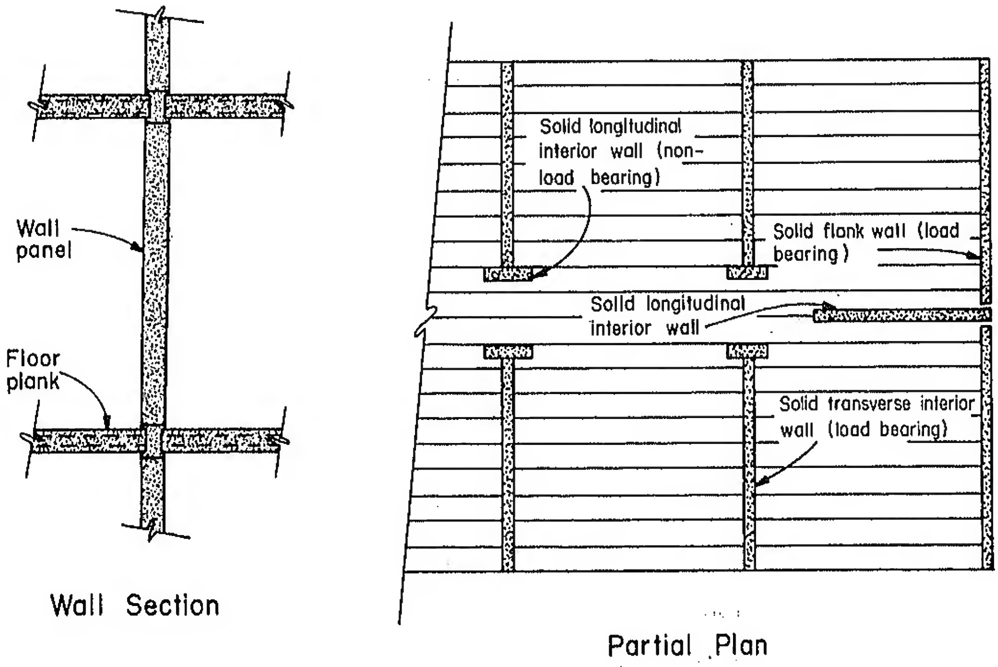


Fig. 4 Large Panel Building Showing Structural Wall Panels in Plan and Section

3.2.2 Production Techniques

Current methods of wall panel production in North America include:

- the battery mold,
- the flat bed or stationary table, and
- the tilt table.

In the battery mold method, the wall panels are cast in arrays (or batteries) of vertical molds, while in the other two methods, the panels are cast in horizontal flat beds. Upon maturity of the concrete and removal of forms or molds, the panels are lifted and moved to storage facilities, prior to shipment to the site. Note that the tilt table method is a variation of the flat bed method and it is used primarily to optimize handling operations.

The engineer should consider the production techniques, especially the lifting and handling operations (these will vary with the plant under consideration), in the selection of reinforcement for wall panels.

3.2.3 Geometry

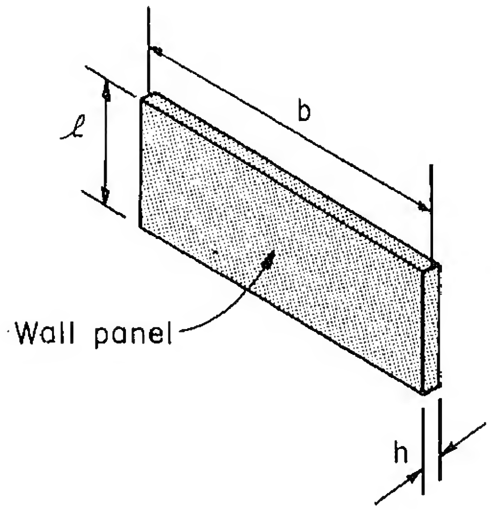
A survey[1] of large panel buildings in North America indicates, that a typical wall panel (Fig. 5), will range in:

(a) length from 10' to 45',

(b) thickness from 6" to 12", and

(c) height from 8' to 10'.

The use of wall panels with thicknesses less than 6" should be carefully investigated for reinforcement placement, bearing details and eccentricity conditions.



ℓ — height, ranging from 8' to 10'

b — length, ranging from 10' to 45'

h — thickness, ranging from 6" to 12"

Fig. 5 Geometry of Typical Wall Panels

3.2.4 Tolerances

Tolerance is the allowable deviation from the specified dimension for the size or shape of a precast element. When the shape or size of a precast element exceeds specified tolerances, a "misfit" occurs. Relative to the wall design, a misfit can result in a change of the location and direction of applied forces and moments, and can cause the creation of unusual restraint conditions.

To avoid misfit between elements it is essential that the engineer specify acceptable tolerances which have been allowed for in design.

A detailed discussion of acceptable tolerances in LP construction was presented in Report 2.[2] For convenience, the recommended tolerances related to wall panels are summarized in Table 1.

TABLE 1 -- RECOMMENDED DIMENSIONAL TOLERANCES FOR WALL PANELS

a. Length	. . ±1/2 in. up to 40 ft; ±1/8 in. additional for each additional 10 ft.	
b. Height	. . ±1/4 in. up to 10 ft; ±1/16 in. additional for each additional 2 ft.	
c. Thickness	. . ±1/8 in.	
d. Out of Square (difference in length of two diagonal measurements)	. . ±1/2 in.	
e. Bowing (convex or concave)	. . 3/8 in. up to 40 ft; plus 1/16 in. for each additional 5 ft not to exceed 1/4 in. in any one 8 ft increment.	
f. Openings (location/size)	. . ±1/2 in.	
g. Position of cast-in items	. . Anchors, weld plates, and inserts	±1/2 in.
	. . Blockouts and reinforcement	±1/4 in.
h. Position of handling devices		±6 in.

3.2.5 Handling Stresses

Prefabricated wall panels should perform satisfactorily during three distinct phases: phase 1--demolding, storage and transportation of components; phase 2--erection; and phase 3--"in-place" condition.

Phase 1--"Demolding, Storage and Transportation of Components"--The adequacy of inserts and the stresses in the wall panel should be checked, taking into account the adhesion of the concrete to the form work, the weight of the partially hardened concrete, the dynamic effects of lifting, and the production techniques. Stresses during storage and transportation should also be considered.

Phase 2--Erection--The stresses in the wall panel should be checked against all conditions of the erection process. The stresses induced in the panels during erection will be effected by such factors as temporary support, bracing conditions, etc. If a "crack

free" wall element is required to resist Phase 3 loadings, tensile stresses in the reinforcement computed under either Phase 1 or 2 loading conditions should be kept relatively low; (say between 12 to 14 ksi for unfactored loadings).

Phase 3--"In Place" Condition--Calculations for this phase form the usual basis for design. The normal loading conditions corresponding to this phase were identified in Report 1.[3]

3.2.6 Loadings

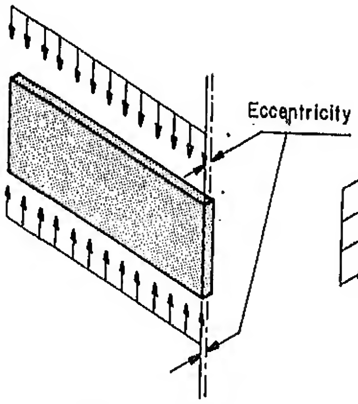
Under normal loadings wall panels are subject to a set of vertical and horizontal forces.

Vertical forces are caused primarily by gravity loads with some contribution as a consequence of lateral loads. Such forces occur in a direction parallel to the middle plane of the wall panel and are typically applied at a specific eccentricity (Fig. 6(a)). Therefore, wall elements under the influence of vertical forces should be designed for the effects of combined flexural and axial loads. In certain cases, instability, which may be aggravated by creep, should also be considered. In flank walls subject to temperature gradients, the effect of gravity loads should be combined with thermal effects.

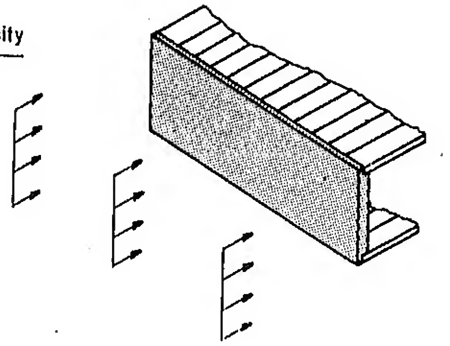
Horizontal forces (Figs. 6(b) and 6(c)) are caused typically by wind or earthquake loads. Such forces occur in a direction either parallel or perpendicular to the middle plane of the wall panel.

Horizontal forces parallel to the plane of the wall panel, while creating nominal peripheral shear stress, can cause significant increase in the axial stress within the wall panel which should be considered in design.

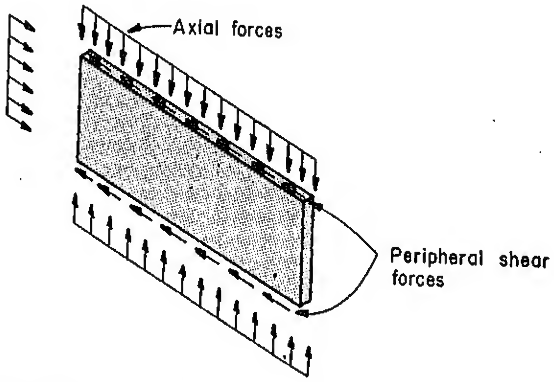
Horizontal forces applied perpendicular to the plane of the wall panel increase the slenderness effects and influence the wall capacity with respect to vertical forces. Such effects occur in flank walls and should be considered in design.



(a) Eccentric vertical forces



(b) Horizontal forces (perpendicular to the plane of the wall)



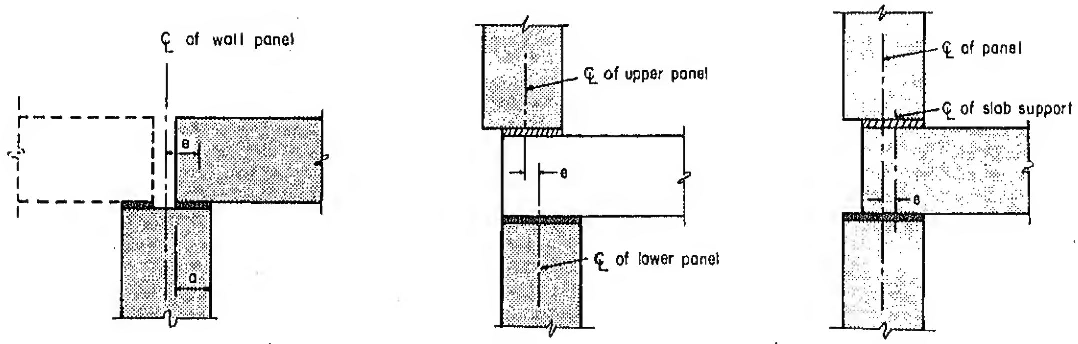
(c) Horizontal forces (parallel to the plane of the wall) along with vertical axial forces

Fig. 6 Vertical and Horizontal Forces on Wall Panels

3.2.7 Eccentricities

In design, eccentricity accounts for the effect of end moments in relation to applied vertical forces. Although the design of most wall panels is usually governed by code specified values[4] for minimum eccentricity, the engineer should be aware of the sources of the various eccentricities.[5]

Structural eccentricities, as illustrated in Fig. 7(a), occur primarily due to the relative positions of the floor and wall elements existing at a connection. Eccentricities may also be



(i) Floor and wall bearing locations

(ii) Non-alignment of joints and panels

(iii) Non alignment of middle planes of successive lifts of panels

Fig. 7 (a) Examples of Structural Eccentricities

generated at the connection either by moment transfer (through adequate detailing) or by clamping action of the upper walls.

Accidental eccentricities (Fig. 7(b)) occur during either the production or erection phase of the wall panel. Accidental eccentricities during the production phase result from (1) differences in the homogeneity of the material, (2) inaccuracies in the thickness of the panel, and (3) lack of flatness. Accidental eccentricities during the erection phase occur due to the panel being out-of-plumb or displaced horizontally from the grid line. It should be noted that the magnitudes of accidental eccentricities change with specified manufacturing and erection tolerances. Note that in the ACI 318-71 Code[4] manufacturing and erection tolerances are assumed limited to a third of the minimum eccentricity requirements.

In practice the range of design eccentricities used in conjunction with to the vertical forces should be based on a combination of structural and accidental eccentricities.

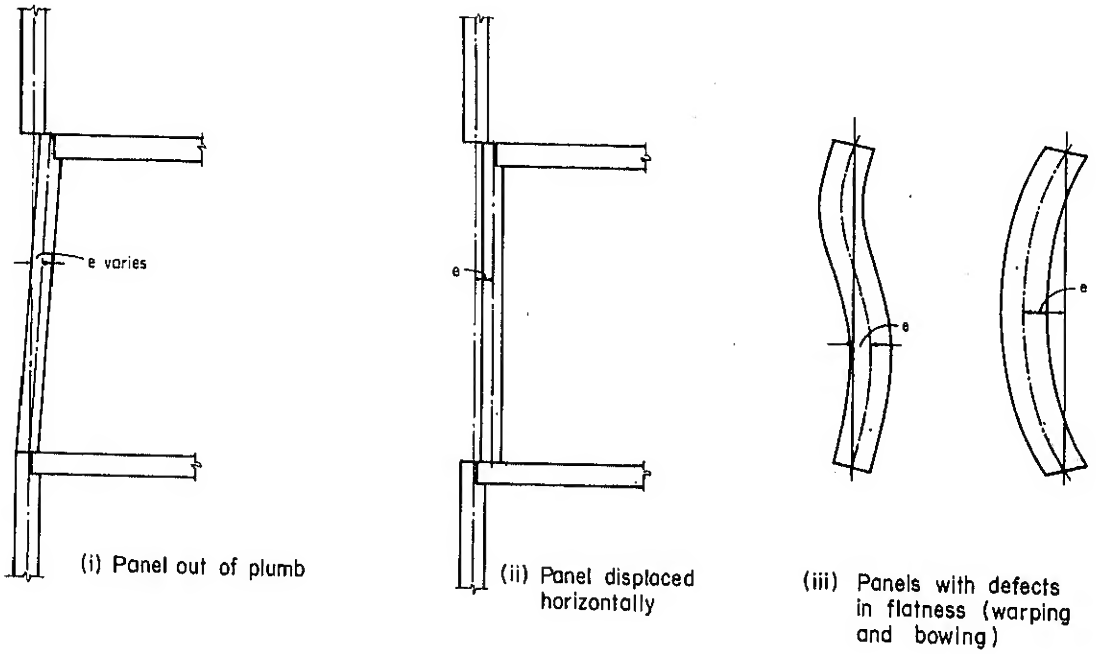


Fig. 7 (b) Examples of Accidental Eccentricities

3.2.8 Reinforcing

Reinforcement requirements for walls vary substantially in the different standards in use today. The British Code of Practice CP 110[5] states that ". . . a wall cannot be considered as a reinforced concrete wall unless the percentage of vertical reinforcement provided is at least 0.4% of the gross concrete cross section."

The PCI considerations for load-bearing wall panel structures[6] suggests a minimum of 0.1% of the gross concrete cross-sectional area for both vertical and horizontal reinforcement. The CEB document[7] requires the provision of reinforcement only if it is needed to support the applied loads.

For a reinforced wall, ACI 318-71[4] requires a minimum of 0.15% and 0.25% of the gross cross-sectional area as vertical and horizontal reinforcement, respectively. However, Section 14.1.2 of the ACI 318-71 Code allows a waiver of these requirements if reinforcement is not needed to satisfy either strength or serviceability. It is important to note that the ACI 318-71 reinforcement requirements does not make a distinction between cast-in-place walls and precast wall panels.

The traditional requirement for minimum uniformly distributed reinforcement in cast-in-place walls of 0.15% of the gross cross-sectional area is primarily an arbitrary value. The historical background for this minimum is based on shrinkage resulting from restraints to which monolithic concrete is subjected. A lower minimum reinforcement (as compared to current ACI 318-71 requirements) is considered sufficient for precast wall panels. A precast panel has little restraint at the edges from the foundation and surrounding structural elements during the curing and storage stages and, therefore, will not build up tensile shrinkage stresses as high as those in the cast-in-place walls.

For purposes of this report (which considers <u>only</u> precast wall panels), a wall panel is considered as <u>uniformly reinforced</u> if (a) vertical reinforcement is required to satisfy strength (load-carrying capacity), and (b) such reinforcement is uniformly distributed and is at least 0.10% of the gross cross-sectional area of the wall.

Wall panels not requiring uniformly distributed vertical reinforcement for strength can be designed as <u>peripherally reinforced</u>.

<u>Uniformly Reinforced Wall Panels</u>--A typical uniformly reinforced wall panel has uniformly distributed vertical and horizontal

reinforcement in addition to vertical tensile ties.* The area of such reinforcement should be at least 0.10% of the gross cross-sectional area in both directions.

Peripherally Reinforced Wall Panels--A typical peripherally reinforced wall panel has vertical tensile ties and horizontal reinforcement at the top and bottom of the panel (Fig. 8). The vertical tensile ties should be sufficient to ensure transfer of vertical forces between two successive stories under conditions of abnormal loadings. Such tensile ties may also be utilized to resist the overturning moment of the multipanel unit and are usually located adjacent to the edges of the panel.

The top and bottom horizontal reinforcement** in combination with the vertical tensile ties helps to prevent crack propagation from the panel edges while creating an ability to act as a simply supported beam. The horizontal reinforcement may be held in place by means of a "ladder" or "truss" type of transverse tie (Fig. 8), which can function to reduce end splitting tendencies in wall panels. The phenomenon of end splitting in wall panels is dependent on the details of horizontal connections and is examined in detail in a subsequent Report: "Connections: Analysis, Design and Acceptance Criteria."

Supplementary Reinforcement--Walls with openings for doors, windows, or mechanical appurtenances should have supplementary reinforcement around the openings to alleviate the effects of high local stress concentrations during handling and under in-place service conditions. It is customary to use the recommendations of ACI 318-71[(4)] for

*Tensile continuity and tie requirements are discussed in detail in Report 2.[(2)]

**For the typical wall panel (8'-0" x 30'-0" x 0'-8"), a minimum of 2 #5 bars (Grade 60) is recommended. This minimum reinforcement is based on the assumption that a wall panel is able to carry its self-weight satisfactorily over a span corresponding to its length.

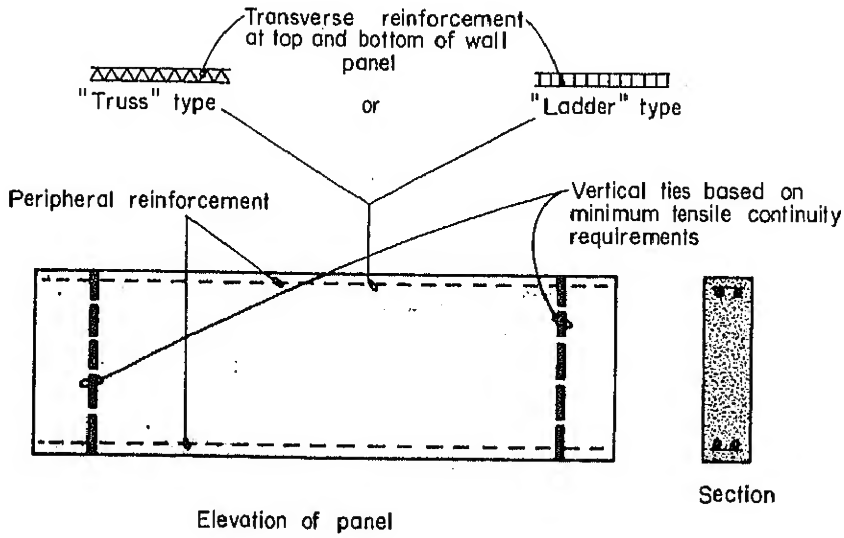


Fig. 8 Typical Peripherally Reinforced Wall Panel

supplementary reinforcement. Portions of the wall which act as lintels or coupling beams must also be suitably reinforced as required by design. At points of significant local bearing stress panels should be reinforced against splitting or crushing.

Ductility--Note that vertical reinforcement exists in both uniformly reinforced and peripherally reinforced wall panels. Regardless of whether such reinforcement is uniformly distributed or concentrated as vertical ties (as in a peripherally reinforced wall panel), it provides a degree of ductility under conditions of abnormal loadings. When severe ductility requirements are necessitated by design (e.g., in a wall panel subjected to earthquake loads), uniformily distributed reinforcement may be needed in all wall panels.

3.3 ANALYSIS

3.3.1 General Procedures for Analysis

The analysis of the overall strength and stability of a wall panel consists of verifying its ability to withstand vertical and horizontal forces, including the effects of buckling and creep. This analysis should, when necessary, be supplemented by an analysis of local effects with regard to concentrated loads.

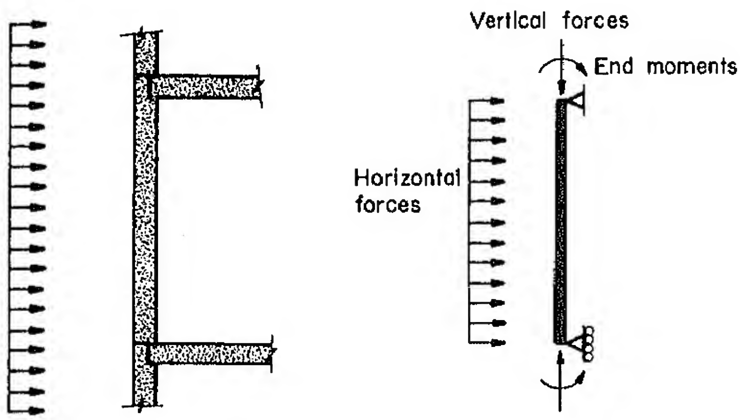
As shown in Fig. 9, wall panels are structural elements subjected primarily to combined flexural and axial loads. If the axial load to which a panel is subjected is not uniform over its entire length, then a strength and stability analysis should be carried out for a critical strip. The determination of the location of the critical strip is dependent upon the vertical edge restraints of the wall panel.

(a) For a wall panel with one restrained edge, the critical strip is located at the free edge of the panel;

(b) For a wall panel with two restrained edges, the critical strip is located at the center of the panel length;

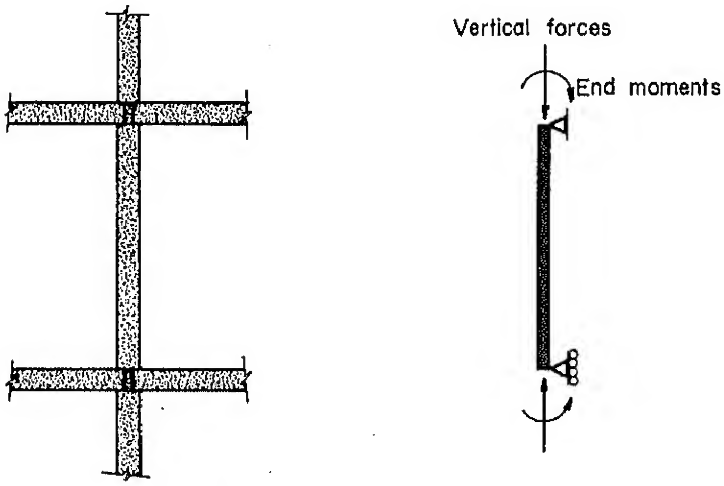
(c) For a wall panel with two free vertical edges, the critical strip is limited by a generating line whose distance from the most compressed edge is equal to the smaller of the following lengths (Fig. 10): 1/3 of the length of the compression zone, or 1/3 of the height of the panel. In this case it is also necessary to analyze the strength of the most compressed edge, not considering buckling.

In practice, the capacity of the panel is established by the capacity of the critically stressed section of the panel.



(a) Actual configuration of solid load bearing flank wall

(b) Analytical model corresponding to (a)



(c) Actual configuration of solid load bearing interior wall

(d) Analytical model corresponding to (c)

Fig. 9 Actual Configurations and Analytical Models for Wall Panels Under Vertical and Horizontal Forces

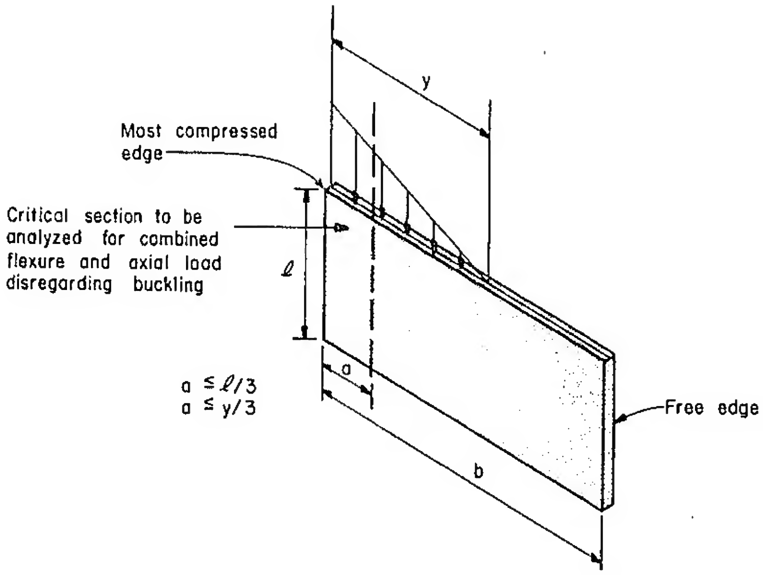


Fig. 10 Location of Critical Strip for a Wall Panel with Two Free Vertical Edges

Strength Analysis

The most convenient method of investigating the strength of concrete sections subject to combined flexural and axial loads is through the use of an "interaction diagram".[8,9] An interaction diagram is a graphic representation of the strength of a section subjected to combined bending and axial loads. Fig. 11 shows a typical interaction diagram. The coordinates of any point A on the interaction curve represent the maximum values of the bending moment, M_t, and axial load, P_t, that can be applied simultaneously on the particular section for which the curve is drawn. The "balanced point", defined by the coordinates (M_b, P_b) on the interaction curve, represents a condition in which the maximum compressive strain in the concrete, ε_u (usually assumed = 0.003 in./in) is reached at the same time at which the strain in the tensile steel, ε_s, reaches the yield strain, ε_y.

The total range of the interaction diagram can be defined by the values of P_t and M_t. Note that the slope of a radial line, P_t/M_t, passing through the origin is equal to the reciprocal of the

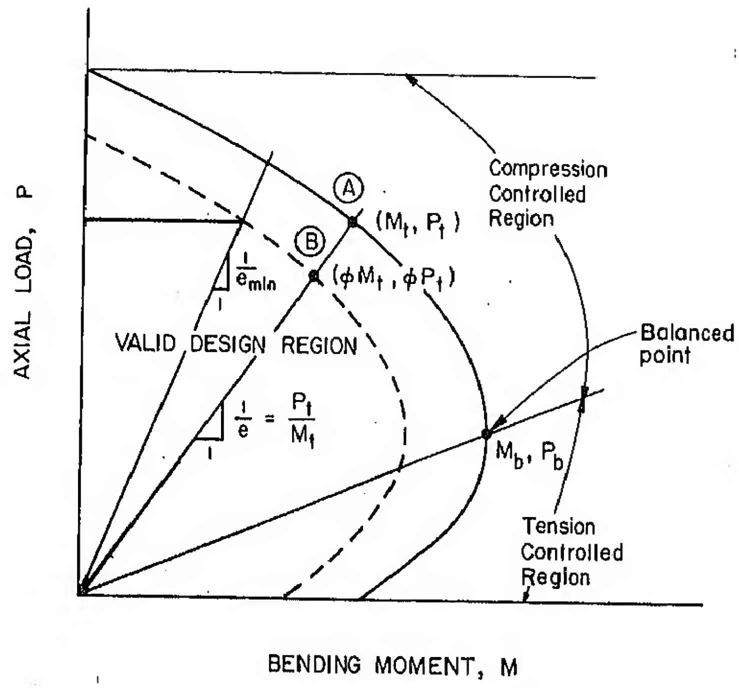


Fig. 11 A Typical Interaction Diagram for a Section Subjected to Combined Flexural and Axial Load

effective eccentricity of the axial load, $e = M_t/P_t$. Thus, lines having relatively flat slopes represent larger eccentricities.

Points on an interaction curve for a particular section are usually obtained by assuming sets of values for the "extreme fiber" strains, i.e., the strain in the extreme fiber of the concrete in compression and the strain in the steel. The extreme fiber compressive strain is set equal to the maximum strain in concrete, ε_u, while the strain in the tensile steel is varied (also see Fig. 12). Points above the balanced point are obtained when the tensile steel strain ε_s, is less than the yield point strain, ε_y, while points below the balanced point are determined when ε_s is greater than ε_y. The forces acting on the section are based on a linear variation of strain between the assumed extreme fiber strains and in accordance with the specified stress-strain relationships for both concrete and steel. The section is usually divided into a number of horizontal strips, and the resultant forces are calculated by numerical

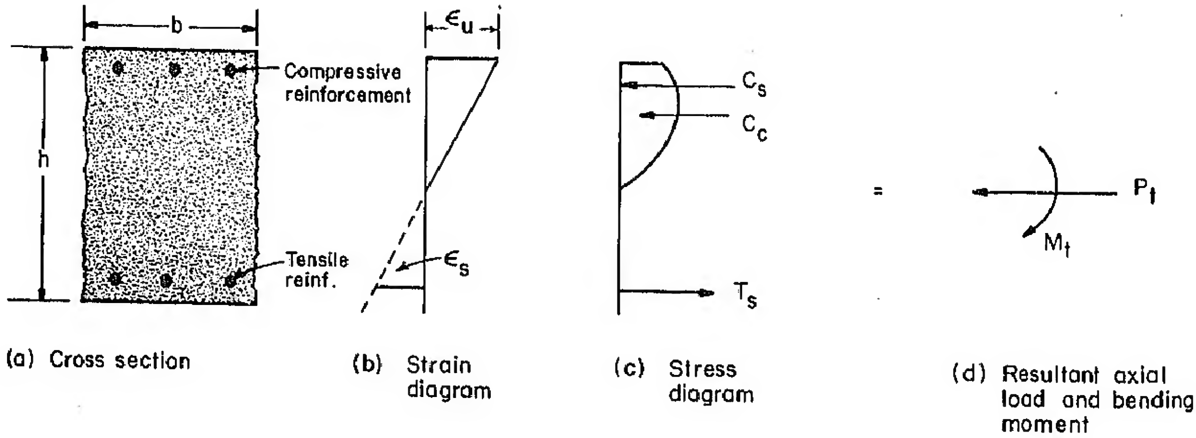


Fig. 12 Distribution of Strain and Stress in a Typical Wall Section at Ultimate Capacity

integration. This process is most conveniently carried out with the help of an electronic digital computer. The use of computer programs[10,11,12] for the analysis and design of sections subjected to combined flexural and axial loads is rapidly becoming common practice.

Under the strength design provisions of ACI 318-71[4] such computed values of ultimate axial load and bending moment must be multiplied by a capacity reduction factor (point B in Fig. 11). This factor provides for the possibility that small adverse variations in material strengths, workmanship, and dimensions, while individually within acceptable tolerances and limits of good practice, may combine to result in undercapacity. Sufficient sectional strength is assured if the design axial loads and flexural moments (i.e., service load times the load factors) fall within the valid design region of the interaction diagram (Fig. 11).

Slenderness Analysis

Under a system of applied vertical and horizontal forces a wall panel should be able to maintain a stable deflected configuration without loss of strength.

In the wall panel illustrated in Fig. 13 the magnitude of bending moments and shear forces depends upon the magnitude of the deflections produced and is sensitive to the eccentricities of the vertical compressive forces.

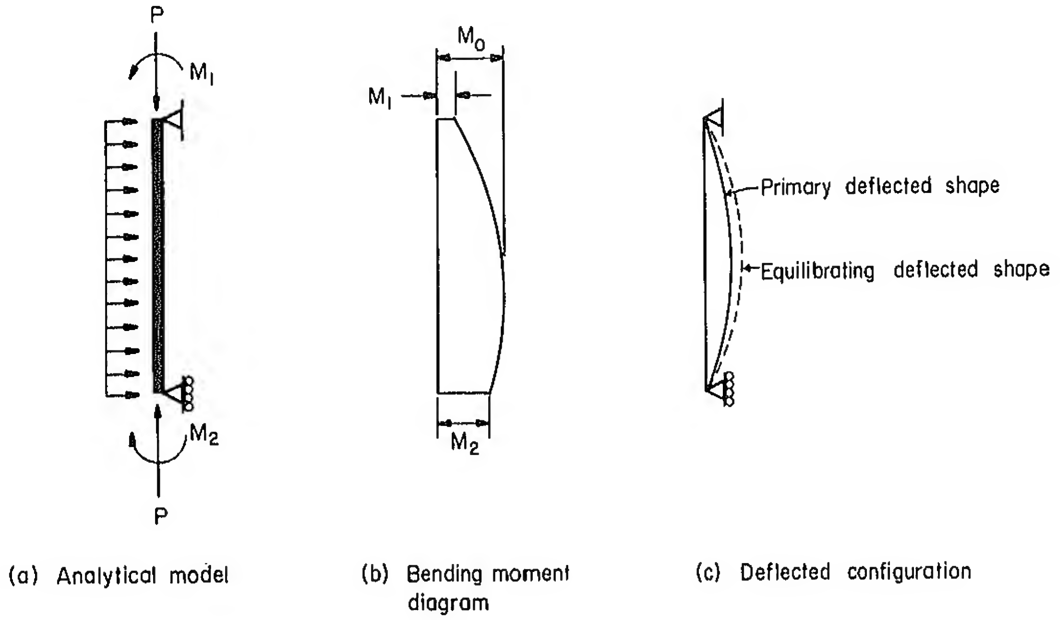


Fig. 13 Basic Principles of the Effects of Slenderness in Stability Analysis

The effect of the vertical forces on the final deflected shape of the panel (i.e., slenderness effects) can be considered a "magnification" (see Fig. 13) of the panel's "primary" deflected configuration. This indicates that the maximum moment, M_{max}, at the critically stressed section can be expressed in general terms by:

$$M_{max} = M_o\delta \qquad \text{(Eq. 3.1)}$$

where:

M_o = maximum moment corresponding to the primary deflected configuration, and

δ = moment magnification factor.

For a truly linear elastic material, the value of δ can be expressed in terms of trigonometric functions.(13) Concrete, however, is not

an infinitely elastic material. Under an applied system of forces concrete cracks, exhibits rheological (creep and shrinkage) characteristics, and follows a nonlinear stress-strain curve. Hence, the exact computation of δ for slender concrete compression members can be an involved process.[14,15,16]

A summary of some of the analytical procedures used in Europe for the design of slender walls is given in Appendix C of this report. However, these procedures cannot be directly adapted to American practice because of the presence of empirical experimental parameters in the design equations.

Simplified design procedures to account for the effects of slenderness can be found in several recommended standards.[4-7,17] One such method discussed in detail in Section 3.3.5 of the ACI 318-71 Code[4] is the Moment Magnification Procedure. In this method the stability of the compression member is achieved by providing sufficient sectional strength to resist the design axial loads and the magnified flexural moments.

3.3.2 Determination of Eccentricities

In computing the resultant end eccentricity for a wall panel consideration should be given to the eccentricity associated with each of the vertical forces. This can be expressed in general terms as:

$$e = (P_t e_1 + P_f e_2) / (P_t + P_f) \qquad \text{(Eq. 3.2)}$$

where:

P_t = vertical force from the wall element,

e_1 = eccentricity of P_t,

P_f = vertical force from the floor elements, i.e., $(P_1 + P_2)$

e_2 = eccentricity of P_f, and

e = resultant end eccentricity.

It should be noted that the magnitude of accidential eccentricities, e_1, is usually accommodated within the minimum eccentricity requirements. The value of e_2 can be determined on the basis of statics. For the typical cases shown in Fig. 14, the value of e_2 can be obtained using the following:

(a) For wall panels loaded on both sides:

$$e_2 = \frac{P_1(h/2 - a_1/3) - P_2(h/2 - a_2/3)}{P_1 + P_2} \qquad \text{(Eq. 3.3)}$$

(b) For wall panels loaded on one side:

$$e_2 = h/2 - a_1/3 \qquad \text{(Eq. 3.4)}$$

where:

P_1,P_2 = vertical forces from floor elements,

a_1,a_2 = bearing widths corresponding to P_1,P_2, and

h = panel thickness.

Knowing the possible maximum and minimum value of accidental eccentricity (e_1), and the structural eccentricity (e_2), the design eccentricity can then be computed using Eq. 3.2. If panels other than a solid type are used, the above equations should be modified to ensure that the eccentricities are measured from the centroidal plane of the load-bearing panel.

3.3.3 Concentrated Vertical Loads

Concentrated loads due to lintel beams, brackets or cantilevers should be distributed uniformly within a zone as indicated in Fig. 15. Where such loads are distributed across vertical connections in panels, the design should provide for the transfer of these loads.[6,17,18]

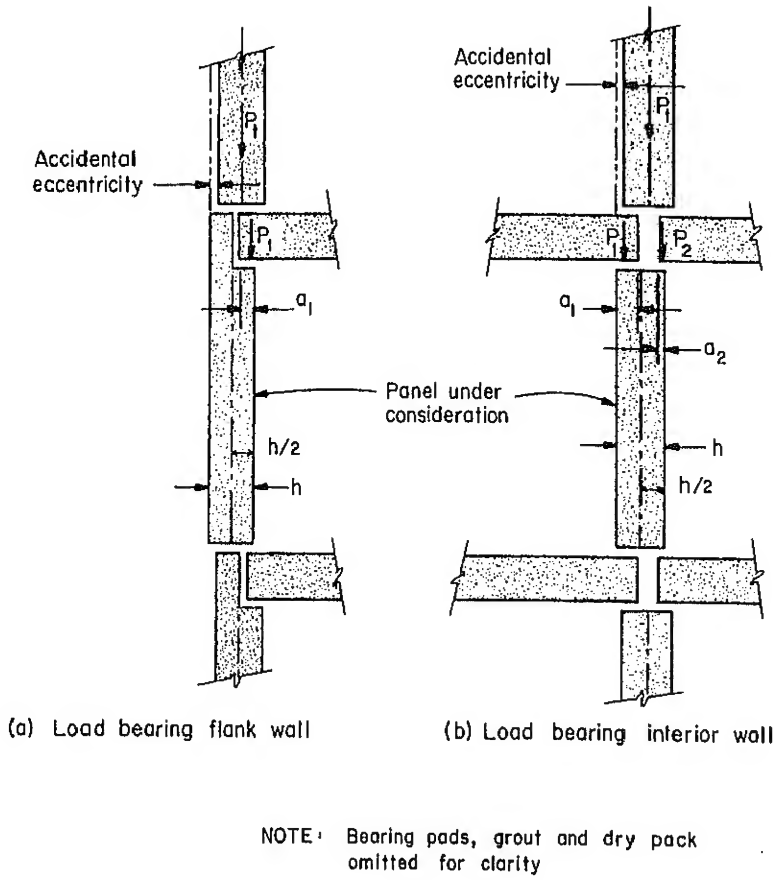


Fig. 14. Determination of Structural Eccentricity (e_2) Due to Vertical Forces from Floor Elements for Typical Wall Panels.

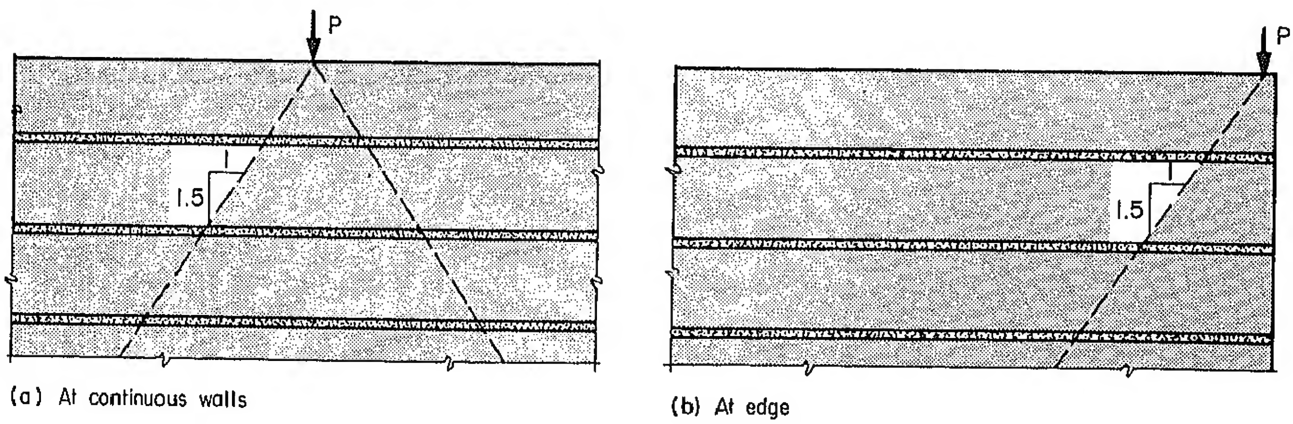


Fig. 15 Distribution of Concentrated Vertical Forces in Wall Panels

Depending on the configuration of the horizontal connection, load concentrations may also occur continuously along the length of the wall panel. If the vertical load transfer between two walls occurs mainly within the cast-in-place concrete, wedge action can cause splitting of the walls at their top and bottom edges. Alternatively, if the vertical load transfer occurs mainly through the ends of floor units, unusually high stresses at the outside edges can cause local compressive failures of the floor or wall units. An extensive study of end splitting in walls is presented in a subsequent report on connections.

3.3.4 Openings and Vertical Loads

As shown in Figure 16, wall panels with large openings for doors, windows, etc., should be proportioned on the basis of the net area effective in resisting vertical loads.

The share of the total load to be supported by the net sections of the wall can be approximated as follows:[18]

$$P_1 = \frac{P(b_1 + 0.5b_o)}{b_1} \qquad \text{(Eq. 3.5)}$$

$$P_2 = \frac{P(b_2 + 0.5b_o)}{b_2} \qquad \text{(Eq. 3.6)}$$

where:

P_1 and P_2 correspond to increased vertical forces on the net wall section.

Care should be taken to relieve stress concentrations around the openings by providing adequate reinforcement. Wall panels with small openings for mechanical penetrations, etc., should also be nominally reinforced around openings to reduce the effects of

locally high stresses. This, however, is not necessary for openings with dimensions less than the wall thickness.

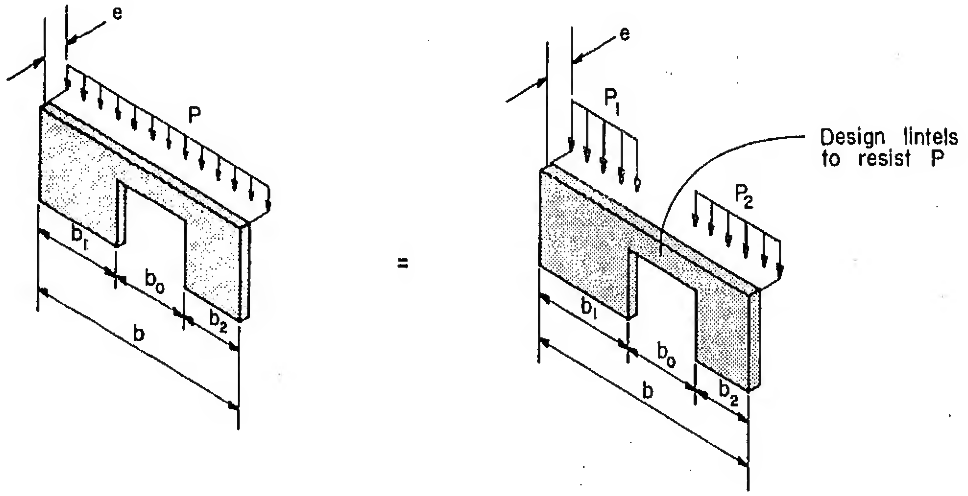


Fig. 16 Equivalent Loadings for Wall Panels with Large Openings

3.3.5 Slenderness Effects

The slenderness of a wall element indicates the degree to which second order deformations are likely to affect (1) the flexural moments acting on critical sections, and (2) the stability of the panel. In general the term "slender" denotes those compression members for which the effects of second order deformations are significant; the term "short" denotes compression members for which such effects may be negligible. Existing standards[4-7,17] define the ranges of slender and short compression members in terms of the slenderness ratio.* Note that the slenderness ratio, $k\ell_u/r$ of

*ACI 318-71 defines the slenderness ratio in terms of $k\ell_u/r$, where "$k\ell_u$" represents the effective length and "r" denotes the radius of gyration. For rectangular compression members, the radius of gyration "r" may be taken as equal to 0.30 times the dimensions of the direction for which the stability is being considered.

typical walls used in LP construction is in the range of 40 to 53 (the corresponding $k\ell_u/h$ values are in the range of 12 to 16), thus falling within the scope of the moment magnification approach of the ACI 318-71 Code.[4]

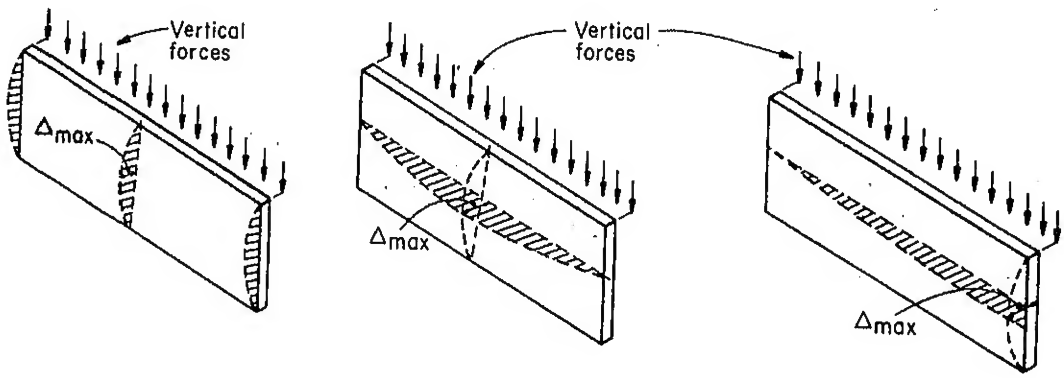
The moment magnification design method of the ACI 318-71 Code is similar to the procedure used as part of the AISC specification[19] for column design. According to the ACI 318-71 Code, the wall design moment computed from a general analysis of the wall element is multiplied by a moment magnifier, δ (see equation 10-5 of the ACI 318-71 Code). The wall cross-section is then designed for the vertical force and the amplified moment. In application, δ is a function of the ratio of the vertical force in the wall panel to the calculated critical buckling load of the wall panel, the ratio of the wall end moments, and the deflected shape of the wall panel.

In computing δ, C_m is an equivalent moment correction factor. The derivation of the moment magnifier assumes that the maximum moment is at or near the midheight of the wall panel. If the maximum applied moment occurs at one end of the wall panel, the design must be based on an "equivalent uniform moment", $C_m M_2$, which would lead to the same maximum moment when magnified.[16]

In defining the stiffness parameter EI for the computation of the critical buckling load, consideration should be given to the effects of cracking, creep, and the nonlinearity of the concrete stress-strain curve. The equations presented in the design methods (see Section 3.4) are based on the experimental and analytical investigations of ASCE-ACI Committee 441[16] and MacGregor et al.[20] It should be noted that the fundamental equation for determining the critical buckling load is based on the assumption of "hinged" ends and must be modified for the effect of end restraints. This is done by using an "effective length", $k\ell_u$, in the calculation of the critical buckling load. The values of the effective length factors and their implications are discussed in the following section.

3.3.6 Effect of Vertical Edge Restraints

A restraint along a vertical edge of a wall panel affects the deflected shape of the panel and hence increases the magnitude of the critical buckling load. For example, a panel which is free along the vertical edges will deflect like a "column" (i.e., flexure primarily in one direction), while a panel which is restrained along either of the vertical edges will deflect like a "plate" (i.e., flexure in two mutually perpendicular directions, as in Fig. 17). To account for the effects of vertical edge restraints, several standards(5,6,7,17) have recommended specific values for the effective length factor, k.



(a) Wall panel with two free vertical edges

(b) Wall panel with two restrained vertical edges

(c) Wall panel with one restrained vertical edge

Fig. 17 Deflected Shapes of Wall Panels with Various Vertical Edge Restraints

Most wall panels in LP construction are braced by orthogonal structural walls. Walls in two directions are used to provide resistance to lateral loads such as wind, earthquake, etc. For such braced wall panels the effective length factor should be estimated on the basis of equations 3.7 - 3.9.(5,6,7,17,18) These

relationships are based on the behavior of wall elements as simply supported plates under the action of vertical forces. Braced wall panels are assumed as hinged at the horizontal connections. The assumption of a hinge is generally conservative.(6,7,18) Assumptions of full fixity can be used for horizontal connections only if detailing methods indicate adequate continuity(6,7,18) and should preferably be supported by tests.

A. For panels braced against sidesway and free along both the vertical edges:

$$\text{for all values of } \ell_u/b, \; k = 1.0 \qquad \text{(Eq. 3.7)}$$

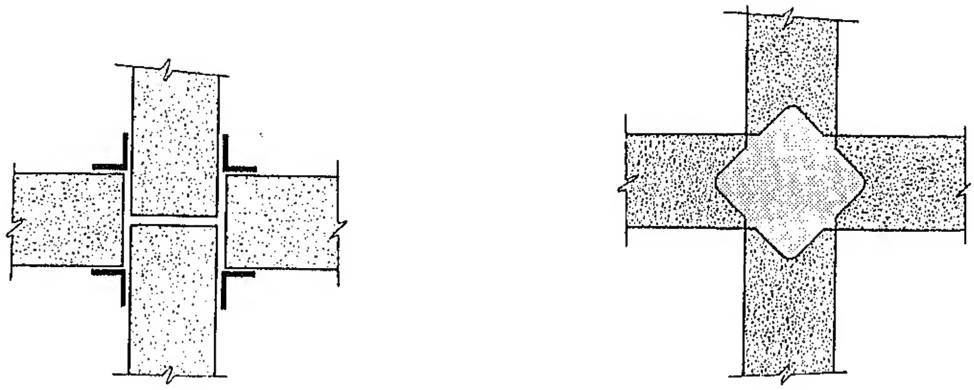
B. For panels braced against sidesway and restrained* along both vertical edges:

$$\begin{aligned} &\text{for } \ell_u/b < 1/2, && k = 1.0 \\ &1/2 \leq \ell_u/b \leq 1, && k = 1.5 - \ell_u/b \qquad \text{(Eq. 3.8)} \\ &\ell_u/b > 2 && k = 1.0/\left[1 + (\ell_u/b)^2\right] \end{aligned}$$

C. For panels braced against sidesway and restrained* along one vertical edge:

$$\begin{aligned} &\text{for } \ell_u/b < 1 && k = 1.0 \qquad \text{(Eq. 3.9)} \\ &1 \leq \ell_u/b \leq 2 && k = 1.0 - 0.423\left[(\ell_u/b) - 1\right] \\ &\ell_u/b > 2 && k = 1.0/\sqrt{\left[1 + 1/2(\ell_u/b)^2\right]} \end{aligned}$$

*A panel is considered restrained along an edge if that edge is prevented from translational displacement perpendicular to the plane of the wall. In vertical edges, this can be accomplished either by thickened edges or by cross walls as shown in Fig. 18.



(a) Cross walls attached by mechanical connectors (b) All four wall edges supported by a cast-in-place joint

Fig. 18 Typical Means (Plan View) of Providing Support at Vertical Edges

Unbraced wall panels are not used extensively in practice in LP construction. The existence of an unbraced wall panel assembly implies that frame action* (i.e., moment transfer between the floor and wall panels) must be developed within the structural system to resist lateral loads. For unbraced wall panels the effective length factor, k, should be determined with due consideration to the effect of cracking and reinforcement on relative stiffness and should be greater than 1.0. A value of k less than 1.2 for unbraced panels would not be realistic.

3.3.7 Thermal Effects

A temperature difference between the faces of a wall panel generally occurs either due to natural seasonal changes or as a consequence of an abnormal loading, for example, a fire. A temperature differential causes a thermal gradient in the wall panel which induces bowing. Simple procedures are available to determine realistic thermal gradients under seasonal changes.[21,22,23,24] However, the computation of thermal gradients under "standard" fire is

*Detailing for frame action will often result in cumbersome and uneconomical horizontal connections and hence should preferably be avoided.

complicated because of changes in the material characteristics with temperatures and due to the transient state of temperature. A detailed investigation of wall panels subject to fire loadings is beyond the slope of this report.

Knowing the steady-state thermal gradient, the bowing effects and the length changes of a wall panel are evaluated using the principles of strength of materials. When one face of a freestanding strip with a length, ℓ, of an infinitely long wall having a temperature, t_1, is subjected to a temperature drop, t_2, $(t_2 < t_1)$, a straight line temperature gradient is established as shown in Fig. 19. The average temperature will then be $(t_1 + t_2)/2$ and the corresponding change in length of the middle plane is:

$$\Delta\ell = [t_1 - (\frac{t_1 + t_2}{2})] \alpha\ell \qquad \text{(Eq. 3.10)}$$

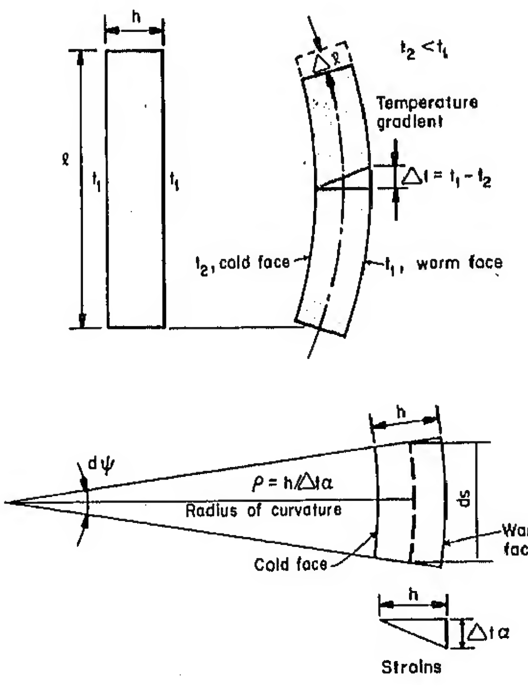


Fig. 19 Bowing and Length Changes of Wall Panel Under a Thermal Gradient

where:

α = coefficient of thermal expansion, and

ℓ = original length of the strip.

The strip also bows as a result of the different temperatures at opposing faces of the wall. The face with the lower temperature shortens as compared to the warm face. The constant angle change along the strip causing the bowing (Fig. 19) is:

$$\frac{\partial \Psi}{\partial s} = \frac{\alpha \Delta t}{h} \qquad \text{(Eq. 3.11)}$$

where:

$\frac{\partial \Psi}{\partial s}$ = angle change,

Δt = temperature differential,

α = coefficient of thermal expansion, and

h = thickness of wall.

From strength of materials it is known that $\frac{\partial \Psi}{\partial s} = \frac{M}{EI}$. By equating the two expressions for $\frac{\partial \Psi}{\partial s}$, the moment required to restrain the strip from bowing can be found by:

$$M = \frac{EI\alpha \Delta t}{h} \qquad \text{(Eq. 3.12)}$$

Unlike the usual structural concept of stresses accompanied by strains as a result of load application, thermal behavior is different and unique. Strains can only be accompanied by stresses if the member is restrained. On the other hand, in restrained members, stresses can occur without strains. For the wall panel shown in Fig. 19, strains exist without stresses. When this element is forced by an external moment M back into its original configuration, it will have stresses but no strains.

Thermal gradients caused by ambient temperature changes (diurnal or seasonal) affect flank walls primarily. These effects can be reduced either by providing sufficient temperature reinforcement or by use of sandwich panels with thermal insulating characteristics.

3.3.8 End Splitting

Longitudinal end splitting of wall panels occurs mainly because of:

(a) transfer of vertical forces in a nonuniform manner (concentrated) through the joint concrete, or

(b) the presence of horizontal forces (perpendicular to the wall panel) at the floor-wall interface.

End splitting causes a reduction in the vertical load-carrying capacity of the wall panel. The magnitude of reduction is, however, dependent to a great extent on the stresses within the horizontal joint.[25]

End splitting tendencies in panels can be reduced either by:

(a) lowering the axial stress level in the panels, or

(b) providing transverse reinforcement at the ends of the panels.

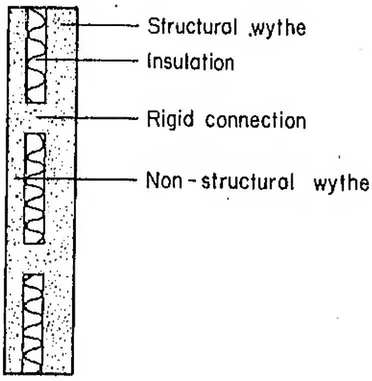
In the latter case such reinforcement may be of either the "trussed" or "ladder" type as discussed in Section 3.2.8. The causes and effects of end splitting are treated in detail with alternate recommendations in Report 5: "Connections: Analysis, Design and Acceptance Criteria."

3.3.9 Sandwich Panels--Types and Considerations

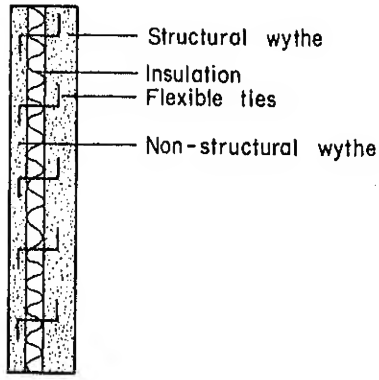
Typically, a sandwich panel consists of two interconnected wythes (layers) of concrete separated by a nonstructural insulation core. The two wythes need not necessarily be of the same thickness. Sandwich panels may be either composite or noncomposite (Fig. 20).

Composite panels

A composite panel has its two wythes connected by concrete ribs or steel shear connectors which restrict the relative movement between the wythes. Under thermal gradients this restraint can result in bowing, stressing, or cracking of panels which affects their performance.



(a) Composite sandwich panel



(b) Non-composite sandwich panel

Fig. 20 Typical Composite and Noncomposite Sandwich Panels

Composite panels are built using a variety of shear ties; e.g., reinforcement used in mortar joints of masonry, expanded metal strips, conventional light gauge steel studs, and concrete ribs. Some of the typical shear ties are shown in Fig. 21. It should be noted that the structural behavior of a composite panel is influenced to a great extent by the type and spacing of its shear connectors.[26]

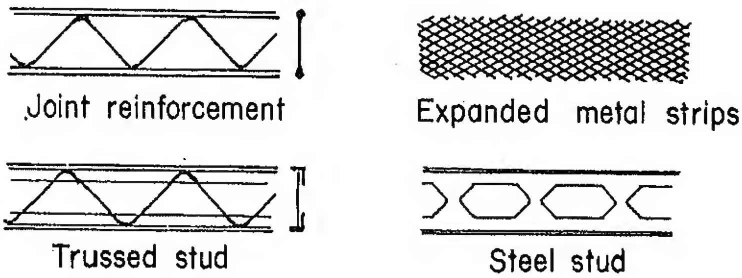


Fig. 21 Shear Connectors for Composite Sandwich Panels

Noncomposite panels

In a noncomposite panel the two wythes are fastened together by flexible ties or hangers. This permits relative movement between the wythes without transfer of stresses. Such flexible ties are generally made of either stainless or galvanized steel. Some common types are shown in Fig. 22.

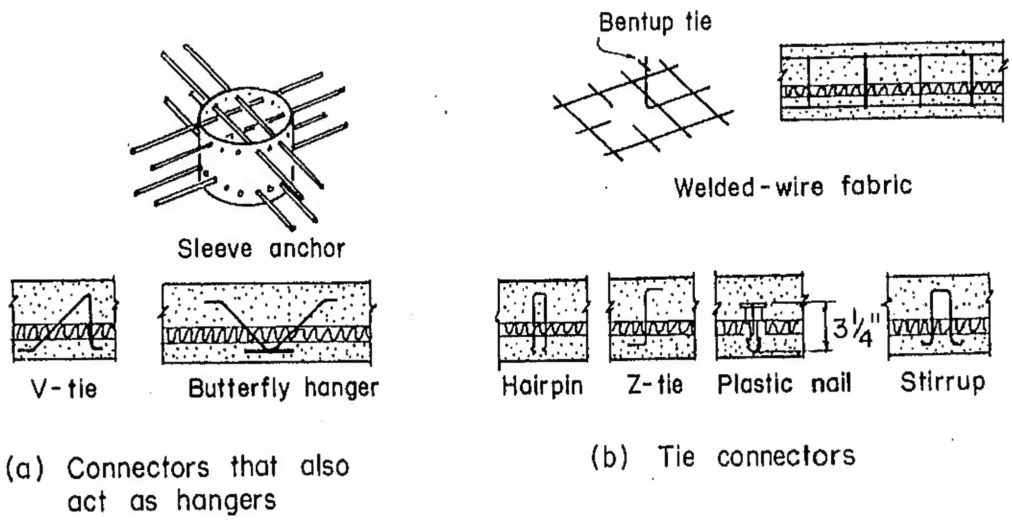


Fig. 22 Connectors for Noncomposite Sandwich Panels

Considerations

In addition to strength and serviceability requirements, the design of sandwich panels(27) should consider the following:

1. Effect of relative movement of the wythes
   The exterior wythe will expand or contract due to temperature differentials between the two wythes. Relative vertical movements can also result from time-dependent effects such as creep and shrinkage.

2. Allowable thermal transmission
   Building codes or standards are moving towards requiring specific U-values* for certain types of occupancy. The thickness of the insulation will affect the total thickness of the panel.

3. Overall weight of the panel
   The geometry of the panel may be limited by the availability of lifting equipment or the flexibility of the panel during the lifting operations.

3.3.10 Wall Panels and Foundation Elements

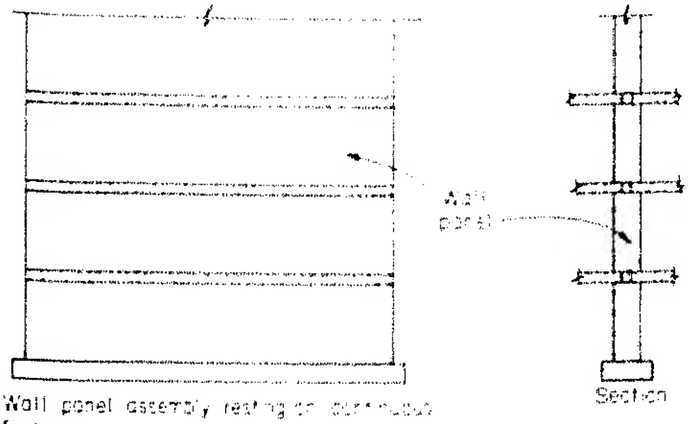
A wall panel assembly in an LP building is generally supported (Fig. 23) in one of the following ways:

(a) continuous footings along the length of the panel, or
(b) isolated foundation elements such as pads, caissons or piers.

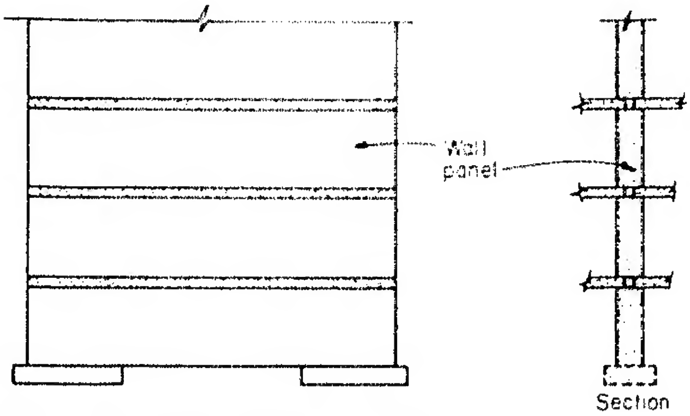
Wall panel assemblies with continuous footings distribute the vertical forces in a uniform manner to the foundations without any bending in the panels themselves. Individual wall panels will behave as independent flexural members in panel assemblies supported on isolated footings. If detailing at horizontal connections provides adequate shear resistance for the panel assembly to behave as a single unit, then a deep beam**behavior will result. Analytical methods to investigate such behavior are available.[28,29,30]

*U-value refers to the thermal conductivity of the element.

**A beam is considered "deep" when it has a height-to-span ratio of 2/5 or more for continuous members, and 4/5 for simple span members.



(a) Wall panel assembly resting on continuous footing



(b) Wall panel assembly resting on isolated footings

Fig. 23 Wall Panel Assemblies and Foundation Elements

3.4 DESIGN

3.4.1 Review of ACI Design Specifications

A review of building codes in the United States by Oberlander[31] indicates that the development of design procedures for load-bearing concrete walls has been at a much slower pace than for other reinforced concrete members.

The Joint Committee on Reinforced Concrete, formed in 1904, consisted of representatives from four national societies: the American Society for Testing and Materials, the American Society of Civil Engineers, the American Railway Engineering and Maintenance of Way Association, and the Association of Portland Cement Manufacturers. The Joint Committee reported that incompatibility existed between theory and tests of reinforced concrete members. Several reports of the Joint Committee were produced (based on research at universities, governmental agencies, etc.) during the period 1909 to 1925. Their final report in 1925 was the first code, a forerunner of the form and content of the Reinforced Concrete Building Code in use today.[4]

Under Section 1109 of the 1928 ACI Code,[32] compressive stresses in walls with an ℓ_u/h ratio of 15 were limited to $0.125f'_c$. A linear reduction was applied beyond this point to $0.0625f'_c$ when the height of the wall was 25 times its thickness. In the 1936 version of the code,[33] the allowable service load compressive stress was increased to $0.20f'_c$ for walls with an ℓ_u/h ratio of 10 or less and decreasing proportionately to $0.11f'_c$ for walls with an ℓ_u/h ratio of 25.

Section 1112 of the 1941 Code[34] increased the allowable working compressive stress to $0.25f'_c$ for walls with a height-to-thickness ratio of 10 or less, and reduced linearly to $0.15f'_c$ for walls having a height-to-thickness ratio of 25. The 1956 ACI Code[35] retained the same allowable stresses and height-to-thickness

requirements as the 1941 Code. The 1963 ACI Code[36] presented the first major change in the design procedure since the original 1928 provisions by introducing the following equation for allowable compressive stress:

$$f_c = 0.225 f'_c \, [1 - (\ell_u/40h)^3] \qquad \text{(Eq. 3.13)}$$

where:

f_c = allowable stress

f'_c = specified compressive strength of concrete

ℓ_u/h = height-to-thickness ratio of the wall.

The above equation resulted from the recommendation that the equation in the Uniform Building Code for allowable compressive stress in reinforced concrete bearing walls be used.[37] That equation, $0.2f'_c \, [1.0 - (\ell_u/30h)^3]$, appeared in the 1943 edition of the UBC published by the Pacific Coast Building Conference (now the International Conference of Building Officials). ACI Committee 318 adjusted the UBC equation to yield results fairly consistent with what had been used by ACI since 1941. The coefficient 0.225 was chosen originally to agree with the coefficient being considered for columns. When the column coefficient was later changed to 0.25, the coefficient for the wall equation was left unchanged.

The reduction of the allowable stress in the 1963 ACI code for short walls from $0.25f'_c$ (which had been used since 1941) to a value of $0.225f'_c$ caused an extensive discussion in the profession. Also controversy existed over the term "reasonably concentric loads" in the 1963 Code equation for the position of the load.

The 1971 ACI Code[4] adopted the "Strength Design Method" as the principal design procedure with the wall design equation given as:

$$P_u = 0.55 \varphi f'_c A_g \, [1.0 - (\ell_u/40h)^2] \qquad \text{(Eq. 3.14)}$$

where:

P_u = factored vertical load on the wall

f'_c = specified compressive strength of concrete

A_g = cross-sectional area

ℓ_u/h = height-to-thickness ratio of the wall

φ = capacity reduction factor.

The ACI 1971 Code defined "reasonably concentric loads" as those applied within the middle third of the cross section. In lieu of Eq. 3-14, the code allowed for the design of wall elements as "columns."

Figure 24 shows the basic design relationships for concrete bearing walls in accordance with ACI codes from 1928 to 1971.

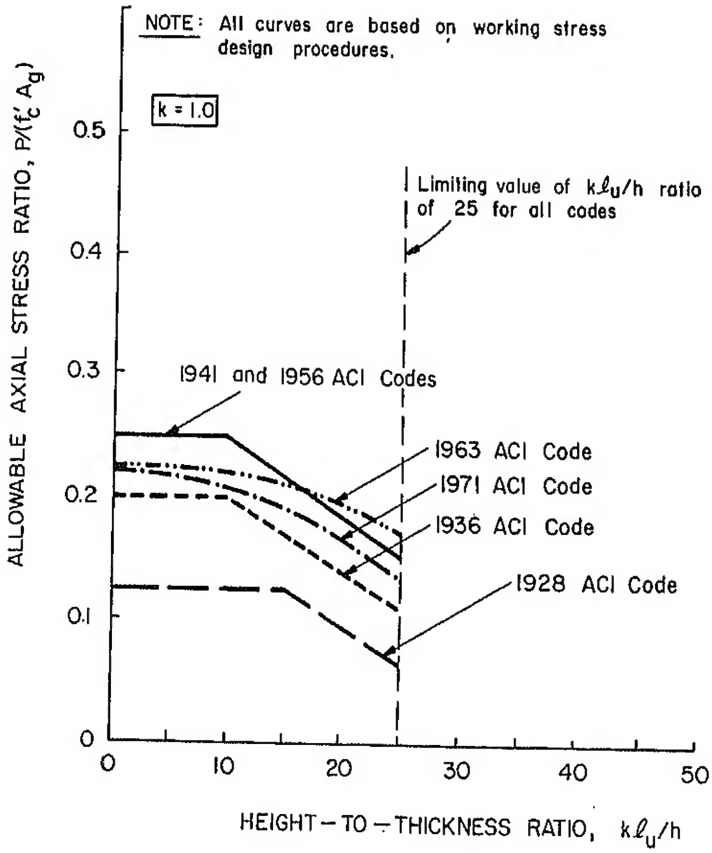


Fig. 24 Allowable Axial Stress in Bearing Walls as Recommended by Various ACI Codes

3.4.2 Uniformly Reinforced and Peripherally Reinforced Wall Panels

The choice of the type of wall panel reinforcement to be used in a particular location of an LP building should be based on strength and serviceability considerations.

Uniformly distributed vertical reinforcement may be required in wall panels to satisfy either strength requirements (e.g., in load-bearing flank walls, where eccentricity may be outside the kern and in lower story interior walls, where axial loads exceed the capacity of unreinforced sections), or serviceability criteria (e.g., in exposed flank walls which are subject to thermal gradients). Wall panels requiring such vertical reinforcement should be designed as "uniformly reinforced" wall panels. Details regarding minimum reinforcement requirements for such wall panels are indicated in Section 3.2.8.

Wall panels may be designed as "peripherally reinforced" only when uniformly distributed vertical reinforcement is not required to satisfy either strength of serviceability. A peripherally reinforced wall panel can be used generally for the majority of interior wall panels in LP residential construction. The axial stress in such panels is of a relatively low magnitude (i.e., less than 22% of f'_c for service loads). Figure 25 shows in schematic form the relationship between span, wall thickness, and number of stories. It also illustrates the ranges for uniformly reinforced and peripherally reinforced solid wall panels. Figure 25 was developed as a guide for preliminary thickness selection on the basis of assumptions listed in Table 2. Details regarding minimum reinforcement requirements for peripherally reinforced wall panels are indicated in Section 3.2.8.

Design methods indicated in Sections 3.4.3 and 3.4.4 below are based primarily on the recommendations of ACI 318-71[4] and ACI 322-72.[38] These methods provide a realistic means of computing the load-carrying capacity of uniformly reinforced or peripherally reinforced wall panels subject to normal loadings.

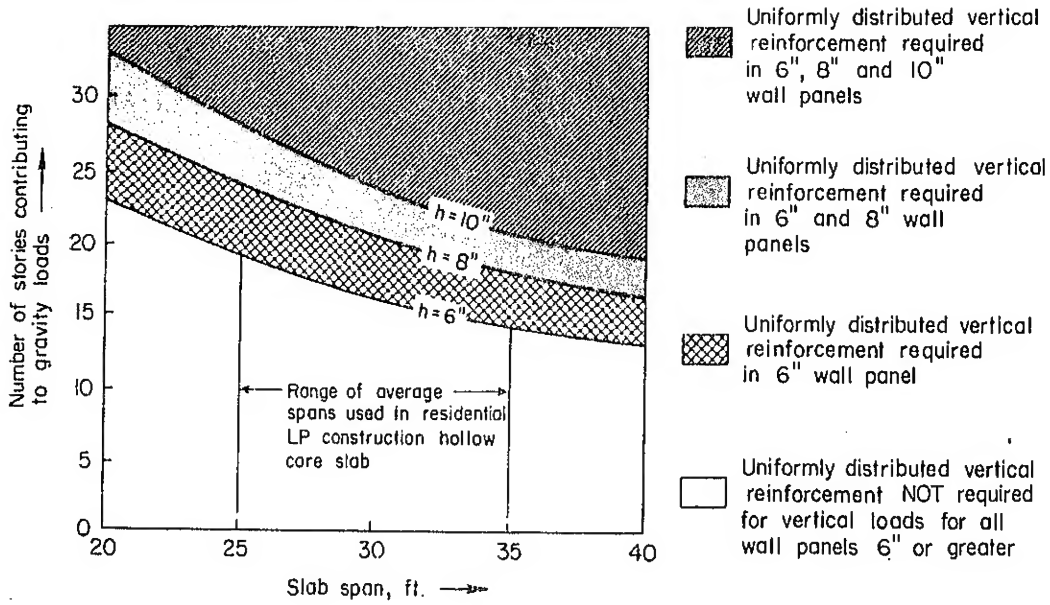


Fig. 25 Schematic relationship between slab span and number of stories for uniformly reinforced or peripherally reinforced interior solid wall panels used in typical LP residential construction. (Note that this graph is based on the assumptions listed in Table 2 and should be used only as a preliminary thickness selection guide.)

TABLE 2 -- ASSUMPTIONS USED IN THE DEVELOPMENT OF THE SCHEMATIC FIGURE 25

1. Dead load (8" hollow core) of floor/roof slabs = 55 psf*
2. Partition load = 8 psf
3. Live load (as per ANSI) = 40 psf
4. Roof live load = 20 psf
5. Story height = 8'0"
6. Interior wall location; wall width = 30'0"
7. Total eccentricities assumed as less than a sixth of the panel thickness.
8. Wall thickness, slab span, and height of building are used as variables.
9. Wall dead load is included and is considered as normal weight concrete.
10. Live load reduction as per ANSI standard.
11. Lateral load effects are considered empirically. The additional axial stress in wall panels due to lateral load is taken as 30% of the total dead plus live load stress.
12. All calculations are for service load conditions.
13. Wall concrete strength, f'_c = 5000 psi
14. The allowable axial stress as per equation (14-1) of ACI 318-71 Code(4) for WSD conditions is:

$$f_c = 0.22 f'_c \left[1 - \left(\frac{\ell_u}{40h}\right)^2\right]$$

$$= 0.22 \times 5 \times .91$$

$$= 1.000 \text{ ksi}$$

15. The horizontal connection is able to transfer the vertical load of the wall.

*The unit weight of 8" hollow core slabs ranges from 53 to 68 psf depending on the width of the unit. The value of 55 psf is considered as representative.

3.4.3 Uniformly Reinforced Wall Panels

Uniformly reinforced wall panels should have uniformly distributed vertical and horizontal reinforcement. A detailed discussion of the reinforcement requirements is presented in Section 3.2.8.

The design of such wall panels for flexural and axial loads should be based on the forces and moments determined from an analysis of the structure. Such an analysis should take into account the effect of deflections on the moments and forces, and the effects of the duration of the loads. To account for such effects in wall panels the moment magnification method of the ACI 318-71 Code[4] should be used.

Strength design of wall panels for combined flexural and axial loads is based on satisfaction of the applicable conditions of equilibrium and compatibility of strains. The relationship between the concrete compressive stress distribution and the concrete strain in reinforced concrete elements may be assumed to be rectangular, trapezoidal, parabolic, or any other shape which results in prediction of strength in substantial agreement with results of comprehensive tests.

Uniformly reinforced wall panels should preferably be braced against sidesway. Braced wall panels are usually assumed to be simply supported along the horizontal connection unless detailed otherwise. The effective length factor, k, is determined on the basis of the restraint conditions existing along the vertical connections (see Section 3.3.6 of this report for specific values of k).

For wall panels not braced against sidesway the effective length factor, k, is determined with due consideration to the effect of cracking and reinforcement on relative stiffness, and is typically greater than 1.2.

For wall panels with a single layer of reinforcement the value of EI for use in Eq. (10-6) of the ACI 318-71 Code[4] may be taken as:[20]

$$EI = \frac{E_c I_g}{\beta_d}\left(0.5 - \frac{e}{h}\right) \geqslant \frac{0.10 E_c I_g}{\beta_d} \qquad \text{(Eq. 3.15)*}$$

For wall panels with a double layer of reinforcement the value of EI for use in the same equation may be taken either as:

$$EI = \frac{\frac{E_c I_g}{5} + E_s I_s}{1 + \beta_d} \qquad \text{(Eq. 3.16)}$$

or conservatively:

$$EI = \frac{\frac{E_c I_g}{2.5}}{1 + \beta_d} \qquad \text{(Eq. 3.17)}$$

For wall panels subject to transverse loading (as in the case of flank walls), the maximum moment can occur at a section away from the end of the member. In this case the value of the largest calculated moment occurring anywhere along the member is used for the value of M_2 in Eq. (10-4) of the ACI 318-71 Code. C_m is taken as 1.0 for this case.

Wall panels designed under the provisions of this section may also require supplementary reinforcement around openings for doors, windows, etc. (For a detailed discussion of reinforcement, see Section 3.2.8).

*This equation is based on preliminary analytical studies indicated in Reference 20.

It should be noted that nonsolid reinforced wall panels are designed on the same basis as solid panels with due consideration given to voids, projections, nonstructural wythes, etc., in determining section properties and strength requirements.

3.4.4 Peripherally Reinforced Wall Panels

Peripherally reinforced wall panels can be used only if (a) the wall panels are concentrically loaded (i.e., when design eccentricities are within the kern), and (b) there are no transverse loadings between horizontal supports. The design of such wall panels for flexural and axial loads should be based on the forces and moments determined from an analysis of the structure.

The axial load-carrying capacity of a peripherally reinforced wall is estimated using equation (14-1) of the ACI 318-71 Code[4] which is nearly identical* with the recommendation of ACI 322.[38] The ACI 318-71 Code equation is modified as follows to take into account the effect of support conditions:

$$P_u = 0.55\varphi f'_c bh \left[1 - \left(\frac{k\ell_u}{40h}\right)^2\right]^{**} \qquad \text{(Eq. 3.18)}$$

In the transverse direction peripherally reinforced wall panels should preferably be braced against sidesway. Braced wall panels are usually assumed as simply supported along the horizontal connection unless detailed to assure a restraint moment. The effective length factor, k, is determined on the basis of the restraint conditions existing along the vertical connections (see Section 3.3.6 for specific values of k).

*ACI 322 recommends the use of 0.50 instead of 0.55 in Equation (14-1) of the ACI 318-71 Code.

**Subcommittee G of ACI 318 is currently considering the use of "32" instead of "40" in Equation (3-18) to satisfy recent experimental results by Oberlander.[31]

Wall panels designed under the provisions of this section should be peripherally reinforced and may also require supplementary reinforcement around openings for doors, windows, etc. (For a detailed discussion of reinforcement see Section 3.2.8).

Nonsolid peripherally reinforced wall panels may be designed using Eq. 3.18, with due consideration given to voids, projections, nonstructural wythes, etc. in determining section properties.

3.4.5 Comparison of Design Methods and Experimental Results

The methods of computation of load capacities of wall panels (discussed in Sections 3.4.3 and 3.4.4) are compared in Fig. 26. It is noted that for the range of wall panels typically used in LP construction, and for reasonably concentric loads, the two methods give comparable capacities.

Investigations in Europe[39-50] of load-bearing walls included extensive experimental and analytical studies. In the United States, however, the experimental studies of walls[31,51,52] have

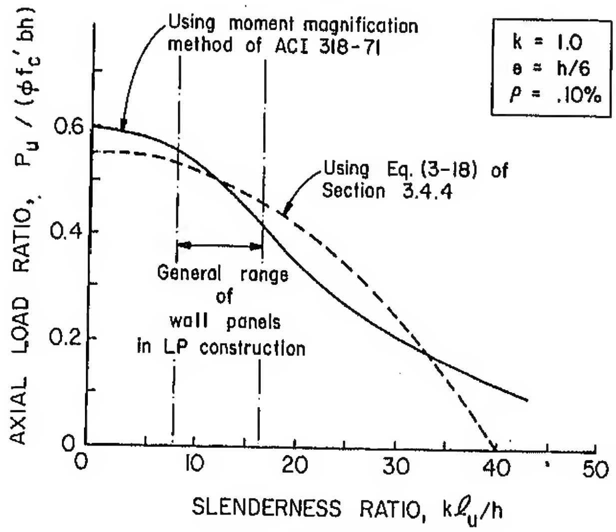


Fig. 26 Comparison of Computed Load Capacities of Wall Panels for Vertical Loads

not kept pace with the analytical procedures and developments for wall design.(53-57)

It should also be noted that available experimental tests of walls, both in Europe and in the United States, are based on end eccentricities less than a third of the wall thickness. No test results have been reported for walls with both vertical and horizontal forces.

Figure 27 shows the comparison of experimental load capacities obtained in PCA tests(52) with the computed capacities using the moment magnification method. It is noted that this design procedure gives realistic and conservative estimates of load capacities for a wide range of slenderness ratios.

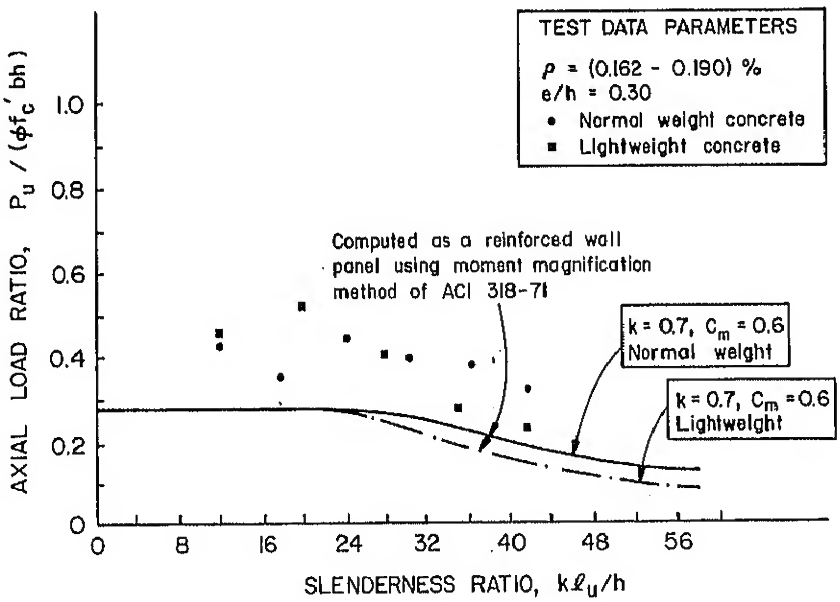


Fig. 27 Comparison of Experimental Load Capacities(52) with the Computed Values Using Moment Magnification Method of ACI 318-71

On the basis of over 50 load-bearing walls, Oberlander(31) also concluded that the moment magnification method of design of wall panels is a satisfactory means of computing the load capacity of lightly reinforced load-bearing walls. Some of the results of his study are shown in Fig. 28.

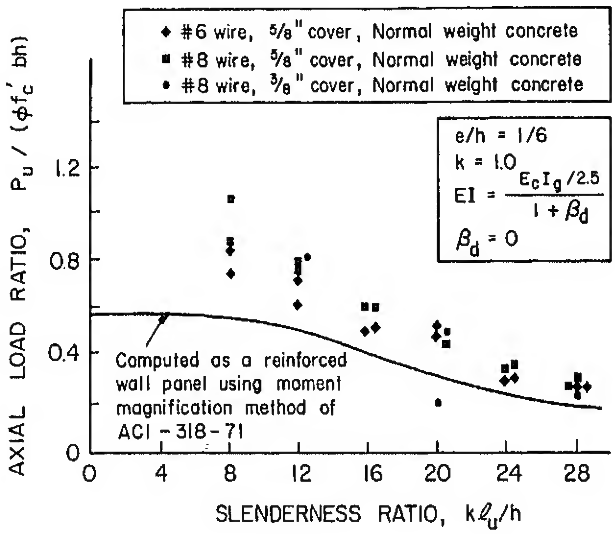


Fig. 28 Comparison of Experimental Load Capacities(31) with the Computed Values Using Moment Magnification Method of ACI 318-71

3.5 SUMMARY AND CONCLUSIONS

Presented in this report is a discussion of the general characteristics of precast wall panels used in residential LP construction. The topics discussed include such items as panel types, production techniques, panel geometry, tolerances, loadings, eccentricities, and reinforcement.

A discussion of the effects of openings, edge restraints, end eccentricities, slenderness effects, end splitting, ductility, and substructure-wall panel interaction is included to assess their relative importance in design. It is concluded that the effects of openings, edge restraints, end eccentricities and slenderness are reasonably significant and should be accounted for in design using the recommendations of ACI[4,38] and PCI.[6]

Wall panels may be designed either as uniformly reinforced or peripherally reinforced elements. The selection of the type of panel to be used in a particular location of a residential LP building should be based on both strength and serviceability considerations. Specific conclusions related to the two types of reinforcement of wall panels are:

1. Wall panels requiring vertical reinforcement (to resist: vertical loads and moments, temperature gradient stresses, stresses due to lateral loads and stresses due to demolding and storage) should be designed as uniformly reinforced wall panels. Such panels should have a minimum of 0.10% of the gross sectional area as vertical and horizontal reinforcement in addition to vertical ties. (A discussion of uniformly reinforced wall panels is contained in Sections 3.2.8, 3.4.2 and 3.4.3).

2. Wall panels not requiring uniformly distributed vertical reinforcement (either from a strength or serviceability viewpoint) and produced in precast plants with good quality control may be designed as peripherally reinforced wall panels. Such wall panels should have vertical ties close to the panel edges and horizontal reinforcement at the top and bottom of the panel. (A discussion of peripherally reinforced wall panels is included in Sections 3.2.8, 3.4.2 and 3.4.4).

The moment magnification method of Chapter 10 and the simplified method of Chapter 14 of ACI 318-71[4] are compared with the results of experimental studies (over 50 tests) and found to be satisfactory.

A comparative study of three common European methods of wall panel design is presented in Appendix C. It is concluded that the European methods cannot be directly adapted to American design practice because of empirical experimental values in the design equations.

Illustrative numerical examples are included in Appendix D to indicate the use of the design methods.

APPENDIX A - NOTATION

a_1, a_2 = bearing distance of floor panels--see Fig. 14

b = length of panel--see Fig. 5; also used as width of compression face of member--see Fig. 12

b_o = length of opening in panel--see Fig. 16

b_1, b_2 = length of segments of a panel--see Fig. 16

e = end eccentricity of vertical forces

e_1 = eccentricity of P_t, see Eq. 3.2

e_2 = eccentricity of P_f, see Eq. 3.2

f'_c = specified compressive strength of concrete, psi

f_s = calculated stress in reinforcement at service loads, ksi

f_y = specified yield strength of reinforcement, ksi

h = structural thickness of wall panel--gross thickness for solid sections, thickness of structural wythe for sandwich panels, and equivalent thickness for ribbed sections

k = effective length factor for compression members

ℓ = nominal vertical dimension (height) of the wall panel

ℓ_e = effective height of panel (to be used in slenderness calculations, $\ell_e = k\ell_u$)

ℓ_u = unsupported height of panel--defined as clear distance between floor panels

r = radius of gyration corresponding to h, r = 0.3h for rectangular sections

t = used as subscript to denote temperature

t_1, t_2 = temperatures on the faces of walls

u = used as subscript to denote ultimate conditions

w = unit weight of concrete, pcf

A = area

A_g = gross area of section

A_s = area of reinforcement

C_m = a factor relating the actual moment diagram to an equivalent uniform moment diagram

E_c = modulus of elasticity of concrete, 33 $w^{1.5}\sqrt{f'_c}$, psi

EI = flexural stiffness of compression members

E_s = modulus of elasticity of steel, psi

I = moment of inertia of cross section

I_g = moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement

I_s = moment of inertia of reinforcement about the centroidal axis of the member cross section

M = bending moment

M_1 = value of smaller design end moment on compression member calculated from a conventional elastic analysis--positive if member is bent in single curvature, negative if bent in double curvature

M_2 = value of larger design moment on compression member, always positive

M_{max} = maximum design moment

M_o = first order bending moment at critical section of compression member as obtained from elastic analysis

M_t = theoretical moment capacity of section corresponding to P_t; $M_t = M_u/\varphi$

M_u = moment capacity of section, corresponding to P_u; $M_u = M_t\varphi$

P = axial load

P_c = critical buckling load of the panel, $P_c = \pi^2 EI/(k\ell_u)^2$

P_f = vertical force from floor element, see Eq. 3.2

P_t = theoretical axial load capacity corresponding to M_t; $P_t = P_u/\varphi$; also used as vertical force from wall element, see Eq. 3.2

P_u = axial load capacity of section, corresponding to M_u; $P_u = P_t\varphi$

P_1, P_2 = axial load in segments of wall panels

α = coefficient of thermal expansion of concrete

β_d = the ratio of maximum design dead load moment to maximum design total load moment, always positive

δ = moment magnification factor

$\Delta\ell$ = increment in length of wall panel due to a thermal gradient

Δ = deflection of wall panel

ε = strain

ε_c = compressive strain in concrete

ε_s = compressive or tensile strain in reinforcement

ε_u = maximum usable compressive strain in concrete

ε_y = yield strain in reinforcement

φ = capacity reduction factor

Ψ = curvature of a section

ρ = reinforcement ratio, A_s/A_g

APPENDIX B
GLOSSARY OF TERMS

Abnormal loading condition:	structurally significant loading condition which is not explicitly included within current codes and standards.
Accidental eccentricity:	eccentricity which exists as a direct result of errors in either the manufacturing or erection process.
Battery mold:	an array of vertical multi-compartment forms for casting wall panels.
Braced wall panel:	a wall panel prevented from translation at the horizontal connections.
Capacity reduction factor:	reduction factor to account for the possibility of small adverse variations in material strengths, workmanship and dimensions.
Composite panel:	sandwich panel connected by concrete ribs or steel shear connectors that restrict relative movement between the wythes.
Critical load:	Euler buckling load of the panel.
Cross wall system:	large panel system in which the load-bearing walls are parallel to each other and perpendicular to the longitudinal axis of the building.

Design methods:	defines either the Strength Design Method or the Alternate Design Method as explained in ACI 318-71.
Ductility:	the measure of a structural component's (element or joint) ability to sustain inelastic deformations, i.e., the ratio of the maximum deformation to the yield deformation.
End splitting:	tendency of the wall panel to split at its ends due to wedge action through the horizontal connection.
Edge restraint:	support conditions at the edges of the panel.
Fit:	this implies proper placement of structural elements (wall panels and floor panels) to ensure the intended load transfer between the elements.
Flat bed:	an array of horizontal forms for casting wall panels.
Flank walls:	those wall elements that are located along the perimeter of the building floor plan.
Floor panel:	horizontal precast concrete element, typically consisting of hollow core precast concrete planks.

General structural integrity:	in generalized terms, the ability of a building to transfer loads from one portion or element which has lost its load-bearing capacity to the surrounding structure while inhibiting progressive collapse and retaining its structural stability; in quantitative terms, it consists of a degree of continuity combined with a degree of ductility within the components and connections of a structure.
Horizontal connection:	the zone common to the wall and floor panels in a horizontal direction.
Interaction diagram:	graphic representation of the strength of a section subjected to combined bending and axial loads.
Interior walls:	those wall elements that are located within the building floor plan.
Large Panel (LP) structures:	structures composed of vertical load-carrying elements of large precast wall panels with precast floors and roofs of panels or planks.
Load-bearing walls:	principal wall elements that carry or distribute vertical and/or horizontal loads to their adjoining structural elements.
Mixed system:	large panel system composed of a combination of cross wall and spine wall systems.

Noncomposite panel: sandwich panel that permits movement between the wythes.

Normal loading condition: loading condition explicitly included in the typical analysis and design of a building by a structural engineer following current codes and practices.

Peripherally reinforced wall panel: wall panel having vertical ties close to the panel edges and horizontal reinforcement at the top and bottom of the panel.

Plank: horizontal precast concrete element, typically extruded and reinforced with high strength steel--also known as hollow core plank or hollow core slab.

Rheological effects: time-dependent effects of concrete such as creep and shrinkage.

Sandwich panel: consists typically of two interconnected wythes (layers) of concrete separated by a nonstructural insulation core.

Service load: unfactored normal loading condition.

Spine wall system: large panel system in which the load-bearing walls are parallel to each other and to the longitudinal axis of the building.

Stability analysis: an analysis which ensures that a wall panel will maintain a stable deflected configuration under a system of applied forces.

Structural eccentricity:	eccentricity which exists as a direct result of a structural detail.
Structural walls:	those walls which, in addition to their own weight, are designed to carry external loads (vertical and/or horizontal).
Structural wythe:	corresponds to that portion of the wall panel that represents the principal load-carrying part of a sandwich panel.
Tilt table:	an array of horizontal forms for casting wall panels that are capable of being tilted.
Traditional loading:	see normal loading conditions.
Tolerance:	the allowable deviation from the specified dimension for the size or shape of the precast element.
Unbraced wall panel:	a wall panel capable of horizontal translation at the horizontal connections.
Uniformly reinforced wall panel:	wall panel having at least 0.10% of the gross area as uniformly distributive vertical and horizontal reinforcement.
U-value:	refers to the thermal conductivity of a material.

Vertical connection: the zone common to two adjacent panels in a vertical direction.

Wall panel: vertical precast concrete element either load-bearing or nonload-bearing. Usually one story in height with lengths typically ranging from 10 to 45 feet.

APPENDIX C

EUROPEAN METHODS OF WALL PANEL DESIGN

C.1 INTRODUCTION

The basis for the CEB method of wall panel design[7] is found in the extensive investigations of Lewicki,[18] Angervo,[48] Larsson[45] and Kukulski.[49] The solutions take into account the nonlinear stress-strain relationship of concrete and partial element weakening caused by cracking of the tensile zone of the cross section. The results of these solutions in the form of nomograms and tables have been put into practice in Europe for the calculation of load-bearing capacity of prefabricated and cast-in-place walls. Their direct adaptation to American practice is inappropriate because of empirical experimental parameters in the design equation.

C.2 LEWICKI'S SIMPLIFIED METHOD[18]

It is assumed that the theoretical load-carrying capacity of a wall, P_t, can be expressed in the form:

$$P_t\ (e,\ell_u/h,\ell_u/b) = \gamma_1(e)\ \gamma_2(\ell_u/h)\ \gamma_3(\ell_u/b)\ A_g f'_c \qquad \text{(Eq. 3.19)}$$

where

γ_1,γ_2 and γ_3 are functions of e,ℓ_u/h and ℓ_u/b, respectively

or

Eq. (3.19) can be further modified as:

$$P_t\ (e,\ell_u/h,\ell_u/b) = \gamma_1(e)\ \gamma_4(\ell_u/h,\ell_u/b)\ A_g f'_c \qquad \text{(Eq. 3.20)}$$

Arrangement of the function $P_t\ (e,\ell_u/h,\ell_u/b)$ in the form shown in Equation 3.20 helps to determine the unknown functions γ_1,γ_2,γ_3 etc. The function $\gamma_1(e)$ is strictly dependent on the geometry of the wall section and the amount of vertical reinforcement in the wall panel. Basically this is a functional relationship of the "interaction" diagram of a section.

For unreinforced rectangular sections, and for eccentricities less than 0.45 h, γ_1 can be estimated by:

$$\gamma_1(e) = 1 - 2e/h \qquad \text{(Eq. 3.21)}$$

With larger eccentricities or for heavily reinforced walls, the function $\gamma_1(e)$ must be computed on the basis of the interaction diagram.

The value of $\gamma_2(\ell_u/h)$ is more difficult to compute and is a function of the buckling stress for the wall panel. Based on an assumed stress-strain relationship for concrete, the function for $\gamma_2(\ell_u/h)$ in terms of the buckling stress is:

$$\gamma_2(\ell_u/h) = \gamma_0/(1 + \gamma_0) \qquad \text{(Eq. 3.22)}$$

where

$\gamma_0 = \pi^2 E_c \, (r/\ell_e)^2$, and

E_c = elasticity modulus of concrete

r = radius of gyration

ℓ_e = effective height of panel, $k\ell_u = \ell_e$

The load-carrying capacity of a column or wall unstiffened along its vertical edges is that of an eccentrically loaded strut, i.e., the effective length factor, k, can be taken as 1.0 provided restraints are not developed along the horizontal edges.

When the wall is stiffened along one or both vertical edges, it tends to behave as an eccentrically loaded plate and the effective length factor, k, is affected. Specific details concerning the value of k were discussed in Section 3.3.6 of this report. The combination of Eq. 3.22 and the relationships for k (see Eq. 3.7 - 3.9) constitute the $\gamma_4(\ell_u/h, \ell_u/b)$ function.

The theoretical load-carrying capacities can then be computed using Eq. 3.20 with appropriate factors of safety.

C.3 LARSSON'S METHOD[45]

Larsson assumed that the deflected shape of a wall panel under eccentric compression is a parabola having its vertex at the middle of the wall. It was also assumed that the flexural rigidity EI varies linearly between the center and ends of the wall panel. It can be shown that the rotational deformation of the concrete cross section as a function of the deflection at the middle of the wall is given by:

$$\varepsilon/\chi = 9.6\ (h/\ell_e)^2\ \delta/h \qquad \text{(Eq. 3.23)}$$

where

ε = compressive strain at one edge

χh = distance from the compressive edge to the neutral axis

h = wall thickness

ℓ_e = effective height of panel

δ = deflection at midheight of panel

Using Eq. 3.23, a trial and error solution* is made to satisfy the equations of equilibrium between the internal stresses and the externally applied loads. The method of computation is involved and consists of the solution of a great number of fourth degree equations.

*The trial and error solution involves the selection of a value of ε and χ, to satisfy a given value of P and δ.

C.4 KUKULSKI'S METHOD[49]

Kukulski's method is an extension of Lewicki's[18] work and also involves the generation of functions γ_1, γ_2, and γ_3. An additional assumption, however, is made concerning the deflection axis as follows:

$$\frac{\partial^2 e}{\partial x^2} = \frac{1}{\rho} = \frac{\Delta\varepsilon}{h} \qquad \text{(Eq. 3.24)}$$

If the difference of external strains $\varepsilon_1 - \varepsilon_2 = \Delta\varepsilon$ is presented as a function of mean stress σ_0 and eccentricity, e becomes a function of σ_0 and e. This function may be considered as $\Delta\varepsilon(\sigma_0 = \text{const}, e)$, because the mean stress σ_0 calculated in relation to the whole uncracked element cross section is constant along the whole element length. The values of $\Delta\varepsilon$ may be calculated relatively easily for a given σ_0 and e.

The above approximation simplifies some of the analytical steps involved in the computation of the generalized γ functions.

It should be noted that nomograms[50] have been developed on the basis of this procedure for the calculation of the load-carrying capacity of wall panels.

APPENDIX D

NUMERICAL EXAMPLES OF WALL DESIGN FOR A TYPICAL LP BUILDING

D.1 DESCRIPTION OF BUILDING

The building under consideration is a five story, two bay structure with the configuration shown in Fig. 29.

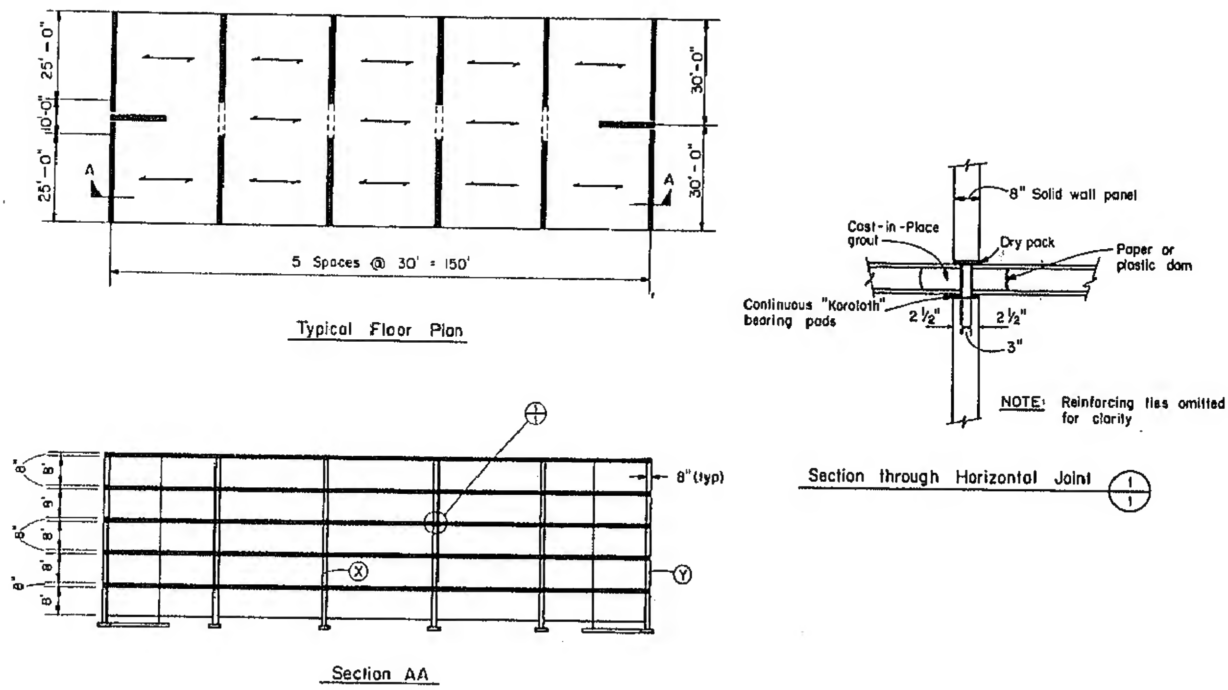


Fig. 29 Plan and Sections Through a Typical Large Panel Building

D.2 DESIGN DATA

It is required to investigate the wall panels X and Y for normal loadings based on the design conditions given below:

Loadings

Floor:	Live load	=	40 psf
	Partition load	=	10 psf
	Mechanical load	=	5 psf
	Slab dead load*	=	55 psf
Roof:	Live load	=	40 psf
	Lateral load	=	20 psf
	Built up roof	=	6 psf
	Slab dead load*	=	55 psf

Materials

Normal weight concrete, f'_c = 4000 psi

Mild steel reinforcement, f_y = 60 ksi

Prestressing strands, f_y = 270 ksi

D.3 CHECK THE LOAD-CARRYING CAPACITY OF PANEL X

Vertical forces on panel

Dead load = (55x30x3 + 8x150x8x3/12 + 61x30) #/ft**
= (4950 + 2400 + 1830) #/ft
= 9180 #/ft

Panel self weight = 8x150x8/12 #/ft
= 800 #/ft

Partition + mechanical load = (15 x 30 x 3) #/ft
= 1350 #/ft

Total dead load, w_d = (9180 + 800 + 1350) #/ft
= 11.33 k/ft

*Slab dead load of 8" hollow core slabs varies from 53 to 68 psf depending on the width. The use of 55 psf is considered as representative.

**#/ft implies pounds/foot.

Live load, w_ℓ* $= (40 \times 30 \times 4)$ #/ft
$= 4.8$ k/ft

Factored load, w ** $= 1.4\ w_d + 1.7\ w_\ell$
$= (1.4\text{x}11.33 + 1.7\text{x}4.8)$ k/ft
$= (15.86 + 8.16)$ k/ft
$= 24.02$ k/ft

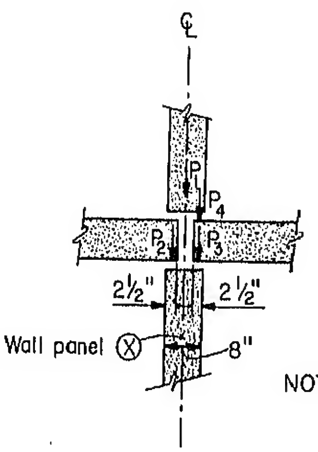
<u>Horizontal forces on panel</u> are absent because panel is at an interior location.

<u>Determination of end eccentricities</u>: eccentricity is determined on the basis of the magnitude and location of the vertical forces P_1, P_2, P_3 and P_4 (Fig. 30). It is assumed that P_1 is transmitted directly into panel X; P_2 and P_3 correspond to dead loads; P_4 corresponds to live load on either span, i.e.:

$$P_1 = [(55 + 15) \times 30\text{x}2 + (55 + 6) \times 30 + 800\text{x}3] \text{ \#/ft}$$
$$= [4200 + 1830 + 2400] \text{ \#/ft}$$
$$= 8430 \text{ \#/ft}$$

$$P_2 = P_3 = (55 + 15) \times 30/2 = 1050 \text{ \#/ft}$$

$$P_4 = 40 \times 30/2 = 600 \text{ \#/ft}$$



NOTE: Tie reinforcing, bearing pads, grout and dry pack omitted for clarity

Fig. 30 Horizontal Joint Through Panel X

*Live load reductions may be used in an actual design situation.

**Based on minimum ACI 318-71 Code provisions. The other load combinations will have to be checked in an actual design situation.

e_{min}(per ACI 318-71[4]) = 0.1 x 8" or 0.6";

use 0.8"

end eccentricity = $\dfrac{\pm 8430(0.8) + 1050(3.167) - 1050(3.167) + 600(3.167)}{8430 + 1050 + 1050 + 600}$

= 0.78" or -0.43"

therefore

design eccentricity, e_d* = 0.8" ← Use

kern eccentricity, e_k = h/6 = 1.33"

Note that $e_d < e_k$

Eccentricity at the bottom of panel X can be shown to be also within the kern.

Slenderness effects

Effective length factor for a panel with two free vertical edges,

$k = 1.00$

Slenderness ratio, $k\ell_u/h$ = 1.00 x 8 x 12/8 = 12

Therefore, the panel to be designed can be considered as in Fig. 31.

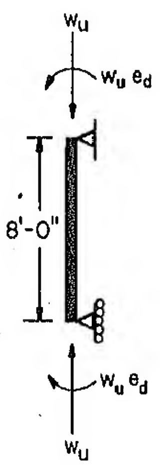


Fig. 31 Analytical Model

*Note the use of e_{min} in the determination of the design end eccentricity.

Capacity computations

Note that panel X is at interior location (hence has no transverse loads between supports) and design eccentricity is within kern; check if a peripherally reinforced wall panel may be used.

The capacity of a peripherally reinforced wall subject to axial loads with eccentricity less than h/6 according to Eq. 3.18 of this report is:

$$
\begin{aligned}
w_{cap} &= 0.55\,\varphi f'_c \times \left[1 - \left(\frac{k\ell_u}{40h}\right)^2\right] A_g \\
&= 0.55 \times 0.70 \times 4.0 \times \left[1 - \left(\frac{12}{40}\right)^2\right] \times 12 \times 8 \\
&= 134.53 \text{ k/ft} > 24.02 \text{ k/ft}
\end{aligned}
$$

Summary

The panel X need not have vertical reinforcement except to fulfill tensile continuity requirements for abnormal loadings. This is explained in detail in Report 2 of this project. The panel, however, should have nominal horizontal peripheral reinforcement, and should also be checked for handling and erection stresses. The final design is shown in Fig. 32.

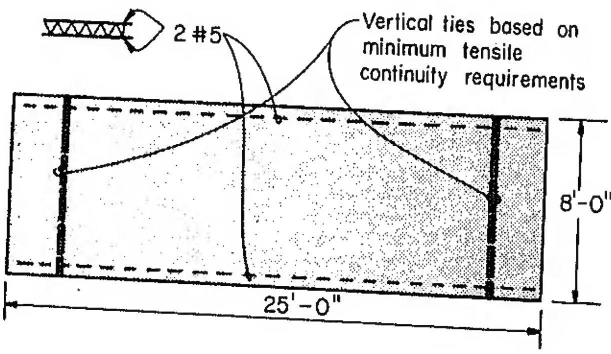


Fig. 32 Peripherally Reinforced Panel X

D.4 CHECK THE LOAD-CARRYING CAPACITY OF PANEL Y

Vertical forces on panel

Dead load	=	(55x15x3 + 8x150x8x3/12 + 61x15) #/ft
	=	(2475 + 2400 + 915) #/ft
	=	5790 #/ft
Panel self weight	=	0.66 x 150 x 8 #/ft
	=	800 #/ft
Partition + mechanical load	=	(15 x 15 x 3) #/ft
	=	675 #/ft
Total dead load, w_d	=	(5790 + 800 + 675) #/ft
	=	7.26 k/ft
Live load, w_ℓ*	=	(40 x 15 x 4) #/ft
	=	2.4 k/ft

Horizontal forces on panel

Wind load, q	=	20 psf

Design forces on panel

Load combination 1: D + L**

w_u	=	$1.4\ w_d + 1.7\ w_\ell$
	=	1.4(7.26) + 1.7(2.4) = 14.24 k/ft
q_u	=	0 k/ft

Load combination 2: D + L - W**

w_u	=	$0.75(1.4\ w_d + 1.7\ w_\ell)$ = 0.75x14.24 k/ft
	=	10.68 k/ft
q_u	=	0.75 x 1.7 x .02 k/ft
	=	0.0255 k/ft

*Live load reduction may be used in an actual design situation.

**Based on principal ACI 318-71 Code provisions. The other load combinations will have to be checked in an actual design situation.

Determination of end eccentricities

Eccentricity is determined on the basis of the magnitude and location of the vertical forces P_1, P_2 and P_3 (Fig. 33). It is assumed that P_1 is tramsmitted directly into panel Y; P_2 and P_3 correspond to dead and live loads, respectively.

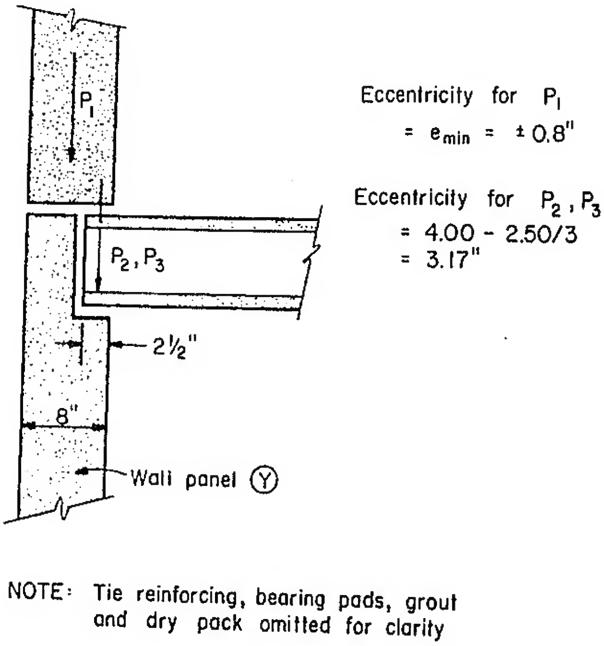


Fig. 33 Horizontal Joint Through Panel Y

$$P_1 = [(55 + 15)15\text{x}2 + 61\text{x}15 + 800\text{x}3] \text{ \#/ft}$$
$$= (2100 + 915 + 2400) \text{ \#/ft}$$
$$= 5.41 \text{ k/ft}$$

$$P_2 = (55 + 15) \times 15 \text{ \#/ft}$$
$$= 1.05 \text{ k/ft}$$

$$P_3 = 40 \times 15 \text{ \#/ft}$$
$$= 0.60 \text{ k/ft}$$

$$\text{end eccentricity} = \frac{\pm 5.41(0.8) + 1.05(3.167) + .6\text{x}3.167}{5.41 + 1.05 + .6}$$
$$= 1.35'' \text{ or } 0.13''$$

$$e_{min} = 0.1 \times 8'' \text{ or } 0.6'', \text{ i.e., } 0.8''$$

design eccentricity, e_d = 1.35" ← USE

kern eccentricity, e_k = $h/6$ = 1.33"

Slenderness effects

Effective length factor for a panel with one free vertical edge ($\ell_u/b = 8/30 < 1$) $k = 1.00$

Slenderness ratio, $k\ell_u/h = 1.00 \times 8 \times 12/8 = 12$

Moment magnifications

Load combination 1:

Using $\beta_d = 0$, $C_m = 1.0$, and $EI = E_cI_g/2.5$, the magnification factor, δ (from ACI 318-71 Code, Chapter 10) is equal to 1.026.

Therefore, the design values are:

w_u = 14.24 k/ft

m_u = $1.026 \times 14.24 \times 1.35 = 19.72$ kin/ft

Load combination 2:

Using $\beta_d = 0$, $C_m = 1.0$, and $EI = E_cI_g/2.5$, the magnification factor, δ (from ACI 318-71 Code, Chapter 10) is equal to 1.019.

Therefore the design values are:

w_u = 10.66 k/ft

m_u (end) = $1.019 \times 10.66 \times 1.35 = 14.66$ kin/ft

Therefore, the panel to be designed can be considered as in Fig. 34.

Minimum reinforcement requirements*

Since the panel is at a flank location it should have a minimum of 0.10% of the gross area as vertical reinforcement and 0.10% of the

*Minimum reinforcement requirements are based on the recommendations of the PCI Report on Precast Concrete Bearing Wall Buildings (Reference 6).

gross area as horizontal reinforcement. Therefore, check whether the minimally reinforced panel is adequate to resist the given load combinations.

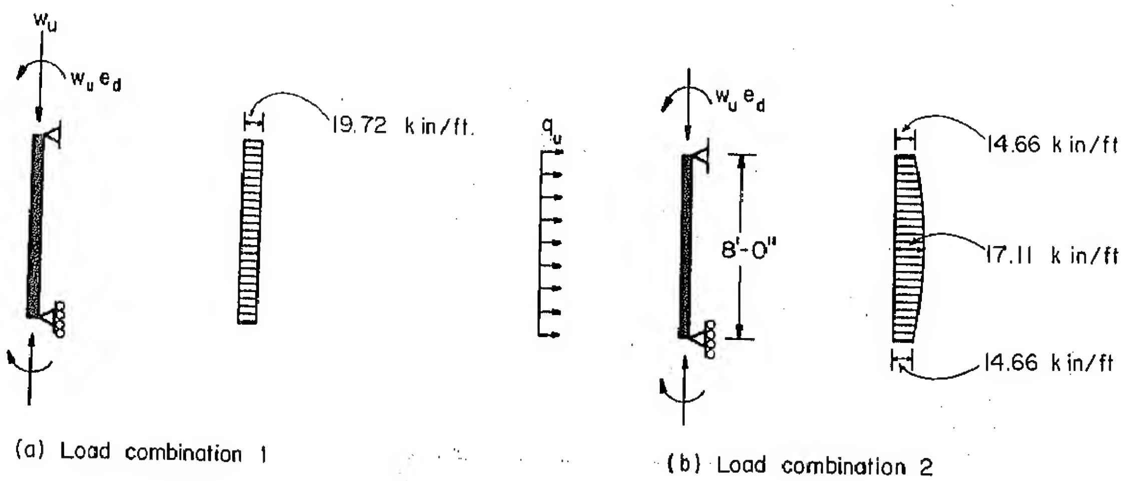


Fig. 34 Load Combinations on Panel Y

Interaction diagram

Based on 0.10% of the gross area as vertical reinforcement, the strength of the wall section is determined using the principles outlined in Section 3.3.1. The resulting interaction diagram (along with the required design load combinations) are shown in Fig. 35. Note that the section has adequate strength and stability to resist the applied normal loads.

Summary

Panel Y should have vertical and horizontal reinforcement (0.10% of the gross section area in each direction). The panel should also have ties for vertical tensile continuity as explained in Report 2 of this project. Handling stresses should also be investigated separately.

The final design is shown in Fig. 36.

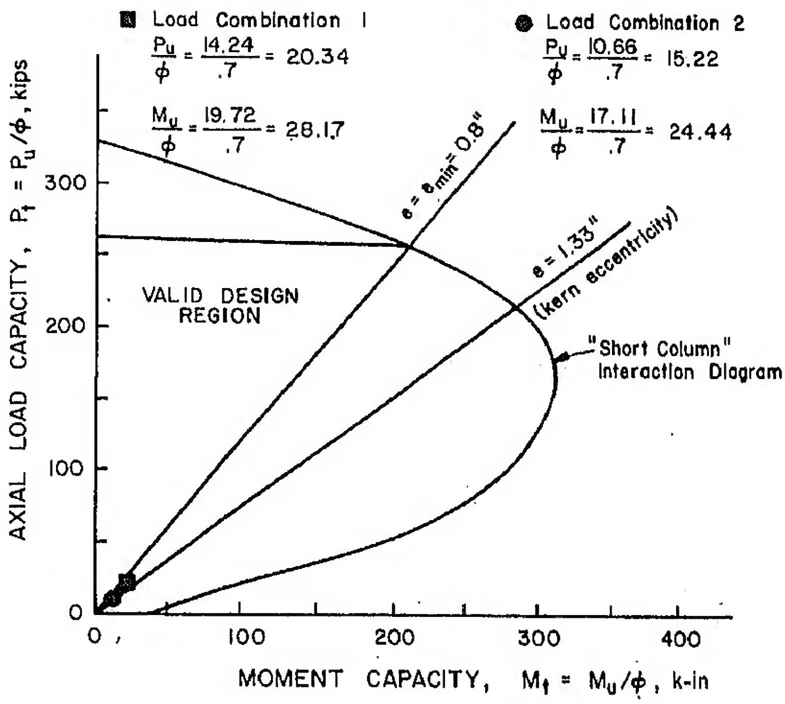


Fig. 35 Design Load and Wall Capacities of Wall Panel Y

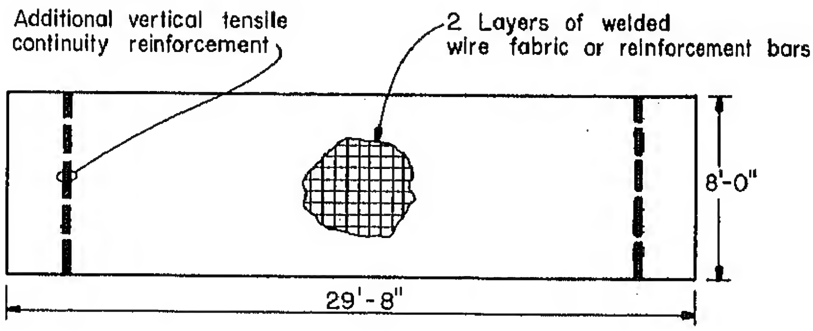


Fig. 36 Reinforced Wall Panel Y

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METRIC CONVERSION FACTORS

The following list will enable readers to convert the U.S. and Canadian customary values of measurements used in this publication to SI (International System) units, the currently recommended form of the metric system. Also included are a few conversion factors that do not conform strictly to SI but are commonly used in some "metric" nations. The proper conversion procedure is to multiply the specified U.S. or Canadian customary value by the conversion factor exactly as given below and then to round to the appropriate number of significant digits desired. For example, to convert 11.4 ft. to meters: 11.4 x 0.3048 = 3.47472 which rounds to 3.47 meters for an accuracy of two significant digits. Do not round either value before performing the multiplication, as accuracy would be reduced. A complete guide to the SI system and its use can be found in ASTM E380, Standard Metric Practice Guide (A Guide to the Use of SI--the International System of Units).

To convert from	to	multiply by
Length		
inch (in.)	centimeter (cm.)	2.54 E*
inch (in.)	meter (m.)	0.0254 E
foot (ft.)	meter (m.)	0.3048 E
yard (yd.)	meter (m.)	0.9144 E
Area		
square foot (sq.ft.)	square meter (sq.m.)	0.09290
square inch (sq.in.)	square centimeter (sq.cm.)	6.452
square inch (sq.in.)	square meter (sq.m.)	0.0006452
square yard (sq.yd.)	square meter (sq.m.)	0.8361
Volume		
cubic inch (cu.in.)	cubic centimeter (cu.cm.)	16.39
cubic inch (cu.in.)	cubic meter (cu.m.)	0.00001639
cubic foot (cu.ft.)	cubic meter (cu.m.)	0.02832
cubic yard (cu.yd.)	cubic meter (cu.m.)	0.7646
gallon (gal.) Can. liquid**	liter	4.546
gallon (gal.) Can. liquid**	cubic meter (cu.m.)	0.004546
gallon (gal.) U.S. liquid**	liter	3.785
gallon (gal.) U.S. liquid**	cubic meter (cu.m.)	0.003785
Force		
kip	kilogram (kgf)	453.6
kip	newton (N)	4,448.
pound (lb.)	kilogram (kgf)	0.4536
pound (lb.)	newton (N)	4.448
Pressure or Stress		
kip per square inch (ksi)	kilogram per square centimeter (kg/sq.cm.)	70.31
pound per square foot (psf)	kilogram per square meter (kg/sq.m.)	4.882
pound (force) per square foot (psf)	pascal (Pa.)†	47.88
pound per square inch (psi)	kilogram per square centimeter (kg/sq.cm.)	0.07031
pound (force) per square inch (psi)	pascal (Pa.)†	6,895.
Mass (Weight)		
pound (lb.) avdp.	kilogram (kg)	0.4536
ton, 2,000 lb.	kilogram (kg)	907.2
grain	kilogram (kg)	0.00006480
Mass (weight) per Length		
kip per linear foot (klf)	kilogram per meter (kg/m.)	0.001488
pound per linear foot (plf)	kilogram per meter (kg/m.)	1.488
Mass per Volume (Density)		
pound per cubic foot (pcf)	kilogram per cubic meter (kg/cu.m.)	16.02
pound per cubic yard (pcy)	kilogram per cubic meter (kg/cu.m.)	0.5933
Temperature		
degree Fahrenheit (deg. F.)	degree Celsius (C)	$t_C = (t_F - 32)/1.8$
degree Fahrenheit (deg. F.)	degree kelvin (K)	$t_K = (t_F + 459.7)/1.8$
Energy		
British thermal unit (Btu)	joule (J)	1,056.
kilowatt-hour (kwh)	joule (J)	3,600,000. E
Power		
horsepower (hp) 550 ft.-lb./sec.	watt (W)	745.7
Velocity		
mile per hour (mph)	kilometer per hour	1.609
mile per hour (mph)	meter per second (m./s.)	0.4470

*E indicates that the factor given is exact.
**One U.S. gallon equals 0.8327 Canadian gallon.
†A pascal equals 1.000 newton per square meter.

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